SETTLEMENT REDUCTION DUE TO SYMMETRIC SHEAR STRESSES BELOW GROUND

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by
N. KUMAR PITCHUMANI

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CERTIFICATE

It is certified that this work entitled "Settlement reduction due to symmetric shear stresses below ground" by N. Kumar Pitchumani (Roll # 8720311) has been carried out under my supervision and has not been submitted elsewhere for the award of a degree.

Dr. M.R. Madhav

Professor
Department of Civil Engineering
Indian Institute of Technology, Kanpur

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NOTATIONS

B	:	Half-Width of Footing
b_s	:	Half-width of the Reinforcing strip
C	:	Depth at which force is acting
G	:	Shear Modulus of soil
i	:	The element at which the influence of the load is required
j	:	The element at which the load is acting
l_s	:	Half-length of the strip
Q	:	Point loading over the footing
S_y	:	Distance of the strip from the centre of the footing
SRC	:	Settlement reduction coefficient
w	:	Vertical displacement of the soil
z_0	:	Depth at which the strip is placed
γ	:	Unit weight of the soil
ν	:	Poisson's Ratio
ν_s	:	Poisson's Ratio of the soil
ρ_z	:	Vertical displacement of the soil
σ	:	Normal stress in the soil
$\tau_{zx}(x)$	:	Shear stress in the x-z plane as a function of x
τ_0	:	The maximum shear stress on the strip
ϕ	:	Friction angle of soil

ABSTRACT

Under circumstances when the soil is soft and weak, soil improvement techniques can be adopted to enhance certain properties of the soil. Earth reinforcement is one such technique. Reinforcements, classified as vertical, inclined and horizontal are being used extensively to improve the performance of the soil. Horizontal reinforcement in the form of geosynthetics are being used under pavements and footings to reduce settlements and improve the bearing capacity. Strip reinforcements are also used widely under footings to improve the soil foundation-response.

An analysis is attempted in this thesis to estimate the reduction in settlement due to an inclusion of strip reinforcement within the soil mass. An elastic continuum approach is used and the solution given by Mindlin (1936) is numerically integrated to study the effect of shear stresses along the strip on the settlement reduction at the surface The question lies in the choice of shear stresses along the strip. The actual distribution of these stresses can be obtained only from a detailed interaction analysis. Since this analysis is beyond the scope of this thesis, five types of shear stress distributions are assumed. Curves have been drawn to show the effect of the length of the strip, distance and depth of placement of the strip on the settlement reduction. Optimum depths of placement have been suggested depending on the length of the reinforcement.

Results obtained from this analytical study have been compared with results obtained from model tests performed by various researchers. Conclusions and limitations of this approach have been given at the end of the thesis.

CHAPTER 1

INTRODUCTION

From time immemorial, when man started off with construction, he has been associated with soil. Any civil engineering structure, whether a compound wall or a multistoreyed building has to be founded in or on the suface of the earth. Soil has been an indispensible item to man. A civil engineer has to acquaint himself with the behaviour and various properties of the soil since he will be dealing with it.

Foundations have to be designed appropriately with proper understanding of the characteristics of the soil below. Once the type of soil and its other properties are known, the type of foundation , shallow or deep is decided by the engineer.

There are situations wherein the soil is extremely weak or soft to take any loads. Under such circumstances, enhancement of the soil properties would result in a significant improvement of the bearing capacity of the soil. With rapid urbanisation, the supply of sites with good foundation conditions has been depleted. These shortcomings call for ground improvement techniques. Various soil improvement methods can be listed down, the latest being the use of geosynthetics. Before venturing into this area it would be reasonable to know the behaviour of unreinforced soil and then look for the improvement brought about by reinforcing the same.

1.1 Unreinforced Soil

When a footing is placed on the soil and a load applied on it, the soil below the footing moves downward and outward.This results in settlement of the footing.

Another important aspect to be studied here is the mode of failure of the foundation and the soil.

1.1.1 Bearing Capacity Failure

As the applied stress on the footing increases, the settlement beneath it increases. Initially the settlements are small and hence can be evaluated using elastic theories. When the maximum shear stress within the soil mass, τ_{max}, equals the shear resistance of the soil then the soil begins to yield. As the applied stress is increased further, the load-settlement curve steepens consequent to the growth of the plastic zone. This condition is called a *local shear failure*. The failure pattern is clearly defined only below the foundation. There is a visible tendency of the soil to bulge on the sides of the footing. However, the vertical compression under the footing is significant and the slip surfaces end somewhere in the soil mass. Only after a considerable vertical displacement of the footing (half width of the footing) may the slip surfaces appear at the ground surface. Even then there is no catastrophic collapse of the footing which remains deeply embedded.

The load-settlement curve steepens gradually until the plastic zone spreads beyond the loaded area. A stage comes when there is a visible decrease of load necessary to produce footing movement. This condition is the *general shear failure*, characterised by the existence of a well defined failure pattern consisting of a continuous slip surface from one edge of the footing to the ground surface. The failure is sudden and catastrophic. A tendency for bulging of adjacent soil can be seen.

For very loose sands, Vesic (1963) has defined a *punching shear failure* wherein the shear zones at the edges of the footing never become well-defined and

no surface heave is observed.

1.1.2 Excessive Settlements

Albeit no rupture is imminent, settlements beyond an *'allowable'* value cause concern. This is because settlements cause the structure to fail structurally even if the foundation is designed with a high factor of safety against shear. Further, the aesthetic appearance of the structure is spoiled if there is an appreciable tilt resulting due to differential settlements.

The term *'allowable'* settlements requires attention here because a residential structure may tolerate an uniform settlement of 25mm to 50mm while certain machine foundations would not tolerate settlements of 0.1mm or even less.

The bearing capacity and settlements require considerable attention in the design of foundations. Enhancement of the soil properties to increase the bearing capacity and hence reduce settlements becomes inevitable due to reasons mentioned earlier. Various techniques such as compaction, preloading, densification, use of geosynthetics can be employed for the same. Geotextiles and geogrids are gaining ground in the form of soil reinforcement so as to improve the bearing capacity and reduce settlements. A discussion on reinforced soils would help unveil the potentials of these inclusions in the soil.

1.2 Soil Reinforcement

Soil improvement can be achieved by reinforcing it with stiffer members of different types. The inclusions used are resisting elements which are generally either linear or plane. The major efforts mobilized in the inclusions can be of four types : *tension, compression, bending, and shearing*. Three types of reinforcing methods can be enumerated, viz.,

1. Vertical reinforcements

2. Inclined reinforcements

3. Horizontal reinforcements using metallic strips, geotextiles and geogrids.

Reinforcement techniques such as stone columns and micropiles can be categorised as vertical reinforcements. A micropile is made of a bar or a tube of a few centimetres diameter surrounded by grout all along its length. The total diameter of a micropile is 10 to 15 centimetres. As pointed out by Viggiani (1981), these piles help in increasing the factor of safety of a slope against sliding and transmit the shear force resulting from the sliding mass to the stable undersoil. Micropiles also help in reinforcing foundation soils.

Stone column is a vertical column of highly compacted sand, gravel and aggregates. The major role of the column is to increase the resistance and the modulus of the foundation soil. It also constitutes a vertical drain. A stone column transfers the excess load coming on the foundation to the underlying soil. Unlike piles, the mechanism of interaction is that of a restrained expansion in the surrounding soft soil. This inclusion is relatively flexible and can therefore withstand essentially compression. However, when the stone column is used to improve the stability of a foundation soil with respect to a general sliding, it also increases significantly the shearing resistance of the reinforced soil.

Inclined reinforcements are used to stabilize slopes and embankments. Steel rods or even wooden planks can be used as reinforcing elements. The failure plane in such cases is usually assumed to intersect the reinforcement.

Horizontal reinforcements in the form of strips and sheets can be used in almost all soil-reinforcement projects. Sheet reinforcement used in highway construction can reduce the thickness of the subgrade. Strips can be used in

embankments and retaining walls. Strips as well as sheets can be used under footings to increase the bearing capacity and reduce settlements. Stresses are distributed over a larger area in case of geosynthetics, thus reducing settlements, especially under footings and pavements.

1.2.1 Mechanisms

Settlement Reduction

Reduction of settlement resulting due to the inclusion of reinforcing elements within the soil mass can be explained as follows.

Consider a reinforced soil-foundation system as shown in Fig. 1.1. As seen earlier, the load coming on the footing will cause the soil below it to move downward and outward, but those soil particles adjacent to the reinforcement will not undergo any lateral movement. So, prevention of the movement of the soil particles on either side of the reinforcement will cause shear stresses to be developed along the reinforcement (Fig 1.1). These shear stresses in turn will cause a heave or an upward movement of the soil above the reinforcement, thus bringing about a net reduction in settlement. It is this aspect of settlement reduction that has been looked into in this research work.

Lateral Restraint

Consider a semi-infinite mass of cohesionless soil at a depth, h.

The vertical stress is

$$\sigma_v = \gamma \cdot h$$

and the at-rest lateral stress is

$$\sigma_h = K_0 \cdot \gamma \cdot h$$

where $K_0 \approx 1 - \sin\phi$

If the soil expands laterally, the lateral stress ($K_0 . \sigma_v$) reduces to the limiting value ($K_a . \sigma_v$).

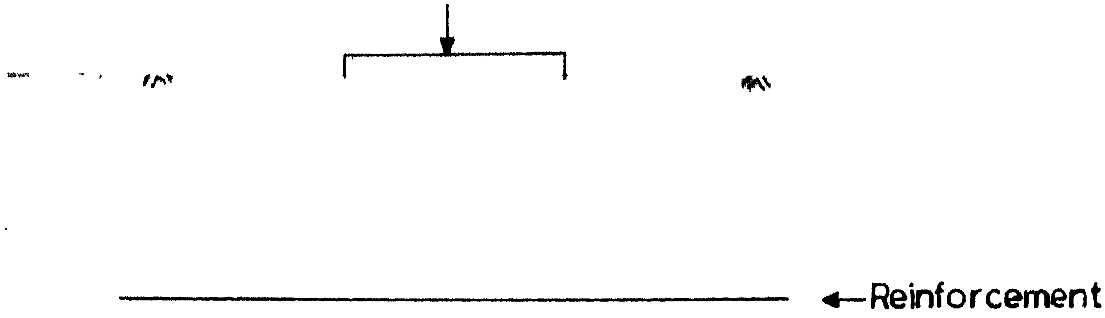

Fig.1.1(a) Reinforced Soil-Foundation System .

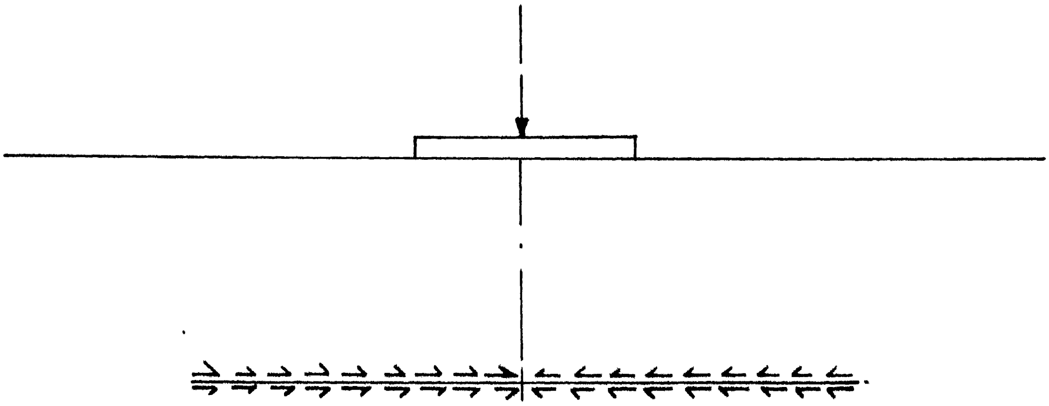

Fig.1.1(b) Stresses on the Soil .

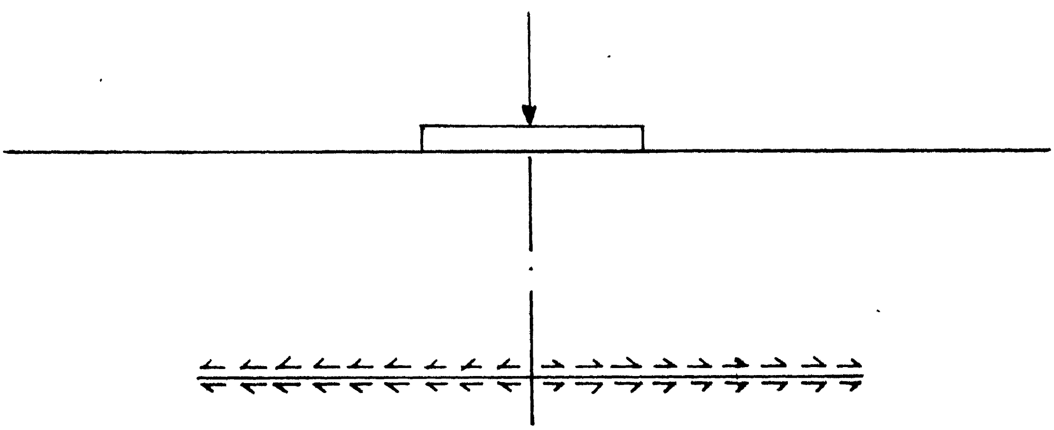

Fig.1.1 (c) Stresses on the Reinforcement.

If a vertical load is applied to the soil, the element gets displaced laterally by δ_h, as well as compressed axially by δ_v. If reinforcement is added to the soil element in the form of horizontal layers, the soil will be restrained, as if acted upon by a lateral force equivalent to the at-rest pressure ($K_0.\sigma_V$), i.e., the effect of the reinforcement is to resist the lateral movement of the soil.

The following chapter reviews the work carried out by various researchers in the field of soil reinforcement. It deals especially with the work attempted towards reducing settlements and increasing the bearing capacity of foundations.

The formulation of the problem under concern is discussed in chapter 3. The elastic continuum approach is employed and numerical integration carried out in determining the settlement reduction due to the stresses acting along the strip reinforcement.

A detailed explanation of the results obtained is given in chapter 4. A comparison with the available data is also attempted. Conclusions and the limitations are summarised in the concluding chapter.

CHAPTER 2
LITERATURE REVIEW

Considerable research has been reported in studying the behaviour of reinforced soil. Since an elastic continuum approach has been resorted to in this thesis, pertinent solutions available in literature are also reviewed.

2.1 Elastic Solutions

Boussinesq (1885) gave the solution for evaluating the normal and shear stresses at any point within the semi-infinite mass due to a force acting on the surface. He has also given expressions for calculating the displacements at any point.

Mindlin (1936) obtained a solution for stresses and displacements at any point within the semi-infinite elastic continuum due to a vertical or a horizontal force acting beneath the surface of the mass at a depth, C.

The solutions given by these two researchers can be integrated within proper limits to obtain solutions for any type of loading such as line, rectangular, strip, circular, linearly increasing, triangular loadings etc. These solutions have been compiled by *Poulos and Davis (1974)*.

Many researchers have conducted laboratory tests to study the behaviour of reinforced soil while some have proposed methods of analysis for the same. A brief review of these studies can bring out the extent of research carried out in this area.

2.2 Model Foundation Studies

2.2.1 Analysis

Binquet and Lee (1975) proposed three possible modes of bearing capacity failures for a foundation having horizontal layers of tensile reinforcement - shear failure above reinforcement, pullout of ties due to slip and tension failure of ties. Using elastic solutions they considered the existence of a maximum shear stress at a distance, x_0, along the length of the reinforcement, x_0 being a function of the depth of placement of the reinforcement. According to them, the maximum shear stress is a function of the loading, q_0, on the footing. *Binquet and Lee* suggested values of τ_{max} as $0.3q_0$ at a depth of 0.25B and around $0.1q_0$ at a depth of 2B, where B is the width of the footing.

Brown and Poulos (1981) used a finite element model to investigate the increase in bearing capacity of a foundation due to the placement of reinforcement in the soil. They compared their results with the experimental data published by Binquet and Lee (1975). They inferred from their results that the quantity of reinforcement necessary to produce a significant increase in the bearing capacity is high, and since the limit state of the reinforcement-soil bond is reached at an early stage, it appears that the area of the reinforcement-soil interface is a significant factor rather than the stiffness of the reinforcement.

Dov and Reinschmidt (1985) presented an analytical approach to study membrane reinforced earth slopes. They used the limit equilibrium and variational extremisation methods. Their results, obtained from closed-form solution indicated that stronger the membrane and more cohesive the soil, deeper the predicted failure. Their results also showed that the presence of a membrane increases the compressive stress over the slip surface and decreases the tensile stress that tends to develop at the crest.

Love, et al. (1987) presented an analysis for the bearing capacity of a layer of fill overlying soft ground. Their results showed that q/C_u for reinforced systems is higher than that for unreinforced systems for the same value of δ/B, where q is the applied load, C_u is the undrained cohesion and δ is the settlement under the footing of width B.

Schmertmann, et al. (1987) made a study using limit equilibrium to predict the number and lengths of reinforcing layers required to stabilize steep slopes of drained frictional fill. They compared the results obtained from this analysis with more detailed analysis methods such as Bishop's modified and Spencer's methods and with previously published results. The design charts presented by the researchers, which are based on a high soil-reinforcement interaction coefficient (μ = 0.9) give reinforcement lengths which are significantly smaller than other published results. There was good agreement with results obtained from detailed analysis. The design charts presented are more practical, yet conservative.

Rowe and Soderman (1987) conducted a finite element study of the behaviour of geotextile reinforced embankments constructed on soils whose undrained strength increases with depth. Their results showed that when the foundation strength increases with depth, the inclusion of high modulus geotextile reinforcement increases the embankment failure heights. They also concluded that tha combination of geotextile reinforcement and strength increase with depth gives rise to a change in the failure mechanism.

Chou, et al. (1987) undertook a study to investigate the effectiveness of placing a single layer of reinforcement at the base of an embankment on a soft foundation. A finite element analysis was resorted to. The settlements obtained from the analysis matched well with the field settlements, both horizontal and

vertical. The use of geogrid did not show much benefit in reducing the differential settlement.

Madhav and Poorooshasb (1988) proposed a new model for reinforced soils. A rough membrane was considered. Combining this element with Winkler springs and Pasternak shear layers to model respectively the soft soils and granular fills, a new foundation model was presented for the geosynthetic-granular fill - soft soil system. Their results showed that the influence of the membrane in reinforcing the soil and reducing settlements is less at small stress levels and significant at higher stresses or on soft soils.

Madhav and Ghosh (1988) presented two and three parameter foundation models to quantify the effect of tensile force on settlements. They also presented solutions for embankment and axisymmetric loading. Their results indicated that in a two parameter model, greater values of tension in the membrane reduce the settlements below the centre of the embankment. They also said that in a three parameter model the maximum settlement reduces and settlements tend to become uniform with increasing tensile force in the membrane and increase in the stiffness of the fill reduces the maximum settlement.

2.2.2 Experimental Tests

Binquet and Lee (1975a) conducted model tests on a 76 mm wide strip footing on the surface of medium dense sand, reinforced with 13 mm wide strips of household aluminium foil spaced to give a linear density ratio, which is defined as the ratio of the total width of the strips to the width of the footing, of 0.425. There was remarkable agreement between theory and experimental data both for the mode of failure and for the bearing capacity ratio (BCR) which is defined as the ratio of the average contact pressures for the reinforced soil and unreinforced

soil. The agreement between theory and experiment was not so good in the case when a soft layer underlay the coarse layer with the reinforcement at the interface.

Joe and Jones (1981) conducted model tests with square footings on a deep homogeneous sand bed reinforced with flat strips of rope fibre material. Their results showed that the bearing capacity ratio decreases with increase in the horizontal spacing of reinforcement. They further concluded that, as the vertical spacing between reinforcements increases, the BCR decreases and when the depth of the first layer is 0.5B, B being the width of the footing, the BCR is maximum.

Verma and Char (1986) conducted tests on model footings on sand subgrades reinforced with galvanised rods placed vertically in the subgrade. Their results showed a considerable increase in the BCR and this improvement, according to them is a function of the diameter of the reinforcements and the spacing of the bars. They stated that, for smaller settlements, the BCR is small and it increases with increase in settlements and concluded that, for a given spacing of reinforcements, the BCR is a function of the diameter and roughness of the bars, while for a given type of reinforcement used, the bearing capacity increases with increasing density of the reinforcements.

Guido, et al. (1985) conducted laboratory model tests to study the bearing capacity of shallow foundations reinforced with geotextiles. Square sheets of geotextile were placed concentrically under the square footing of width equal to 31 cm. Their results showed that at smaller deformations, the full benefits of the presence of the fabric are not exhibited They said, for s/B greater than 0.017, where s is the settlement and B, the width of the footing, the fabric deforms sufficiently to mobilise its tensile stress thereby increasing the load carrying

capacity. For a given value of the depth of the top reinforcement, u, as the vertical spacing of the reinforcements, Δz, decreases, the BCR increases. As the number of layers, N, increases from 0 to 3 there is a steady increase in the BCR, however, as the number of layers increases above three, there is little further change in the BCR. For given values of u/B, Δz/B and N, an optimum width ratio exists which yields the optimum BCR.

Guido, et al. (1987) conducted laboratory model plate loading tests to study the bearing capacity of geogrid reinforced earth slabs. The results they obtained were similar to their earlier work.

Milligan and Love (1984), using laboratory tests studied the mechanism by which the inclusions of a geogrid may improve the performance of unpaved roads and similar construction. Their experimental set up consisted of a box of 1000 mm X 300 mm X 600 mm in dimension Tensar geogrids were placed over a fill of koaline and aggregate placed over the grid and compacted. Their results indicate that geogrids improve the performance of the footing.They concluded that failure loads are about 40% higher than for unreinforced systems and heave in the clay is almost twice as great in the unreinforced system.

Dembicki and Alenowicz (1988) performed model tests on two layer subsoil consisting of a layer of sand fill on the surface of soft clay and investigated the influence of geotextiles on the bearing capacity of the system. Their results showed that the influence of the fabric is significant at large displacements of the footing, but nearly negligible at small deformations. They also concluded that the failure mechanism of the system changes due to the presence of geotextiles.

Patel (1988) conducted laboratory studies by placing fibreglass-geotextile at some depth in a weak soil deposit with sand above the geotextile. His results

showed that the effectiveness of the reinforcement is pronounced if the depth of the placement is 1.5B, where B is the width of the strip footing. If the level of placement of the geotextile is less than 1.3B, the geotextile will act as a rigid boundary and the performance of the sand bed will deteriorate. Increase in this depth beyond 1.65B is not likely to imrove the performance of the foundation any further.

Sreekantiah (1988) conducted tests on square and strip footings on soil reinforced with aluminium foil strips. His experimental investigations revealed that the maximum value of BCR is obtained for u/B ranging from 0.3 to 0.4, where u is the depth of placement of the top reinforcement and B is the width of the footing. He also stated that, as the number of layers of reinforcement increases, the ultimate bearing capacity increases. There was good agreement between experimental results and analytical solutions obtained on the basis of the analysis presented by Binquet and Lee (1975)

Shankariah and Narahari (1988) conducted model footing tests on a sand bed reinforced with three types of reinforcements, viz., G.I.strips, bamboo strips and mild steel welded mesh. Their results showed that the load carrying capacity and stiffness of the sand bed increases by a factor of 1.4 to 4 due to the introduction of reinforcement. The improvement in bearing capacity practically vanishes if the depth of reinforcement is greater than 1.4B, B being the width of the footing. Increase in the number of reinforcing layers increases the bearing capacity and the stiffness of the foundation.

Sargunan and Hussain (1988) conducted model studies to understand the behaviour of loaded footings on reinforced soil using G.I. strips, sand coated G.I. strips, bamboo strips and chicken mesh. Their results indicated a considerable

increase in BCR in the range of 1.47 to 2.85 for various types of reinforcements used. They concluded that chicken mesh would be more effective if placed in two layers.

Narayana and Chandrashekar (1988) conducted tests using synthetic fibres to improve the carrying capacity of lateritic soils. They used 1%, 2%, and 3% fibre by weight of dry soil and conducted the test on a circular footing 150 mm in diameter. The test results indicated that the ultimate bearing capacity increases with 1% and 2% fibre. It is strange to note that 3% fibre inclusion causes a sudden decrease in the bearing capacity value. The elastic settlement decreases with 1% and 2% fibres whereas it increases with 3% fibres.

Sridharan, et al. (1988) presented an analysis of a reinforced sand mattress on soft soil which essentially consists of evaluating the BCR. This analysis was examined with model tests. Their results indicated that by placing the reinforcement at the mid depth of the sand layer, the load carrying capacity substantially increases. The behaviour changes from general shear failure conditon for sand alone to strain hardening nature with reinforced sand mattress. They also infered that the increase in the load carrying capacity with sand mattress of two layers of reinforcement is more than three-fold over that of sawdust, ie., a low friction angle material. Predicted values of BCR demonstrated very close agreement with experimental values.

Bishnoi and Char (1988) conducted a study of the soil behaviour under flexure due to the introduction of geotextiles. They performed tests on beams of rectangular cross-section with varying amounts of reinforcements. They concluded from their results that the introduction of strip form of geotextile reinforcement while imparting ductile failure, only marginally improves the ultimate load carrying

capacity and flexibilty of soil beams. Improvement in load carrying capacity and flexibility is more for deeper beams

Dembicki and Jermolowicz (1988) conducted experiments on a two layered soil medium, made up of silty mud and fine grained sand. The soil was reinforced by a horizontally arranged geotextile layer in plane strain state. Their results showed that a positive effect of reinforcement on the bearing capacity is witnessed for H/B=0.5 to 1, where H is the depth of the reinforcement and B is the width of the footing. The increase in bearing capacity is pronounced at larger strains.

Carroll, et al. (1987) conducted a programme consisting of repeated load tests on varying thickness of reinforced and unreinforced granular bases. Their results indicated that grid reinforcement reduces permanent deformations in flexible pavement systems. Pavement sections incorporating polymer grid reinforcement in the granular base layer can carry three times the number of load applications as conventional unreinforced pavements. Reinforcement allows upto a 50% reduction in the thickness requirements of granular base. The optimum location of the grid reinforcement is at the bottom of thin bases and at the middlepoint of bases 10 inches thick or greater. The researchers also gave design charts using the results obtained.

2.2.3 Field Tests and Case Studies

Fowler and Koerner (1987) summarised a number of soft-soil stabilization projects using geosynthetics. Their emphasis is on the basic use of soil embankment on in-situ soils with shear strength as low as 1KPa. They considered various case histories and arrived at design concepts . They pointed out the specific use of geosynthetics in hastening consolidation and reducing settlements.

Bergado and Sampaco (1988) presented an alternative to pile foundations on soft ground by use of reinforced gravel. They conducted full scale field tests on sewage pipeline reinforced foundation. Data obtained from pull out tests conducted in the laboratory were used in a finite element model on actual culvert construction on reinforced gravel foundation. They concluded from the experiments and analysis that the use of reinforced gravel foundation to support sewage pipeline leads to a reduction in differential settlements and cheaper construction costs. Doubly reinforced gravel yeild better results than geocells.

Kinney (1987) conducted a series of field tests to determine the viability and practicality of geosynthetics to support roads across voids. A design procedure was established and three field tests were run to verify the design procedure. The tests verified the design procedure and suggested that commercially available geosynthetics can be used to support unpaved roads across voids in excess of 3 m wide. He also said that it would be viable to pave the road under certain conditions.

Walls and Galbreath (1987) discussed a case study on the use of geosynthetics to stabilize and reinforce ballasted track over unstable subgrade. The project involved the repair of a section of track in Alabama where a combination of weak soils and high groundwater table had caused repeated track failures. From the case study it was observed that high strength, rigidly structured polymer geogrids prove to be effective and economical in reinforcement of railway ballast to minimize or prevent track instability problems. Properly sized grids will interlock with and confine ballast rock thereby resisting lateral and vertical deformations under typical railroad track loads. Reinforcement of ballast reduces the magnitude of shear stresses transmitted to the subgrade. When

saturated conditions are present, polymer grids will be most effective when used in proper conjunction with appropriate geotextile fabrics that provide adequate separation between subgrade and ballast.

2.3 Shear Strength of Reinforced Soils

Jewell and Wroth (1987) conducted direct shear tests on reinforced samples. Their results showed that a maximum increase in shear resistance is observed when the reinforcement is placed at an angle of 30^0 with the vertical. There is an appreciable increase in the shear stresses due to the presence of reinforcement at higher shear displacements.

Donald and Talal (1986) conducted triaxial compression tests to compare the stress-strain response of sand reinforced with continuous, oriented fabric layers as opposed to randomly distributed discrete fibres. Their test results showed that both types of reinforcement improve strength and increase the axial strain at failure. Fabric reinforcement placed at spacing/diameter ratios greater than one have little effect on strength Discrete, randomly distributed fibres increase both the ultimate strength and stiffness of reinforced sand. Rougher fibres tend to be more effective in increasing the strength.

Lafleur, et al. (1987) conducted direct shear tests to evaluate the adherence between geotextiles and various soils with respect to the design of reinforced lateritic gravel embankments laid over a soft clay foundation. Their test results indicated that the adherence of the fill material to woven geotextiles, is strongly dependent on the grain size distribution of the laterites. When the fines content is less than 10%, the adherence angle δ, can be as high as 44^0, corresponding to an efficiency ($\tan\delta/\tan\phi$) of 85%. When fines content is around 30%, the efficiency drops to 60%.

Krishnaswamy and Raghavendra (1988) conducted direct shear tests on different soils such as kaoline and silty clay soil specimen reinforced by geotextiles. Their results showed that, at low water contents the geotextile does not have significant effect on kaoline but produces a loss in strength in silty clay. At higher water contents the strength increases with the number of reinforcing layers and normal stress. Reinforced samples can take higher strain until failure than unreinforced samples.

Krishnaswamy and Srinivasulu (1988) conducted triaxial compression tests under undrained condition on silty clay soil specimens, reinforced by a commercially available geotextile. Their results showed that at low strains the reinforcement produces a loss in compressive stiffness and this loss is pronounced when the number of layers is more and the moulding water content is low. The strength of the specimen increases as the number of reinforcing layers increase and decreases with increase in water content.

As can be seen from the various studies, there isn't any analysis of reinforced beds using the elastic continuum approach. This thesis makes use of this approach in modelling a homogeneous reinforced soil-foundation system.

CHAPTER 3
FORMULATION AND ANALYSIS

3.1 Definition of the problem

It is proposed to evaluate the reduction in settlements under a footing due the presence of a reinforcing strip within the soil mass.

A strip of negligible thickness, length, $2l_s$, and a relatively small width, $2b_s$, is considered. The strip is placed under the footing, parallel to the width, 2B, at a depth, Z_0, and at a distance, S_y, along the y-axis from the centre of the footing.

3.2 Formulation

As explained earlier in chapter 1, the movement of the soil along the strip causes shear stresses to be developed at the interface of the strip and the soil. It is these stresses that are responsible for the settlement reduction. The crux of the problem lies in formulating a model that will try to explain this function of a reinforcing strip.

An elastic continuum approach has been adopted to solve the problem The width of the strip is assumed to be small enough with the result that the stresses (τ_{yz}) in the y-direction are negligible (Fig. 3.1). Only the shear stresses, $\tau_{zx}(x)$ act along the x-direction of the strip. The present goal will then be to estimate the vertical displacements of the points A, B, etc., (Fig. 3.1) on the surface.

Mindlin (1936) has given a solution for obtaining the vertical displacement of a point, A(x,y,z), due to a horizontal force, Q (Fig. 3.2) acting at a depth C in a semi-infinite elastic continuum as

$$\rho_z = \frac{Q\ .\ x}{16.\pi.G.(\ 1-\nu)}\ .\ [\ F\ ] \qquad \ldots\ (\ 3.1\ )$$

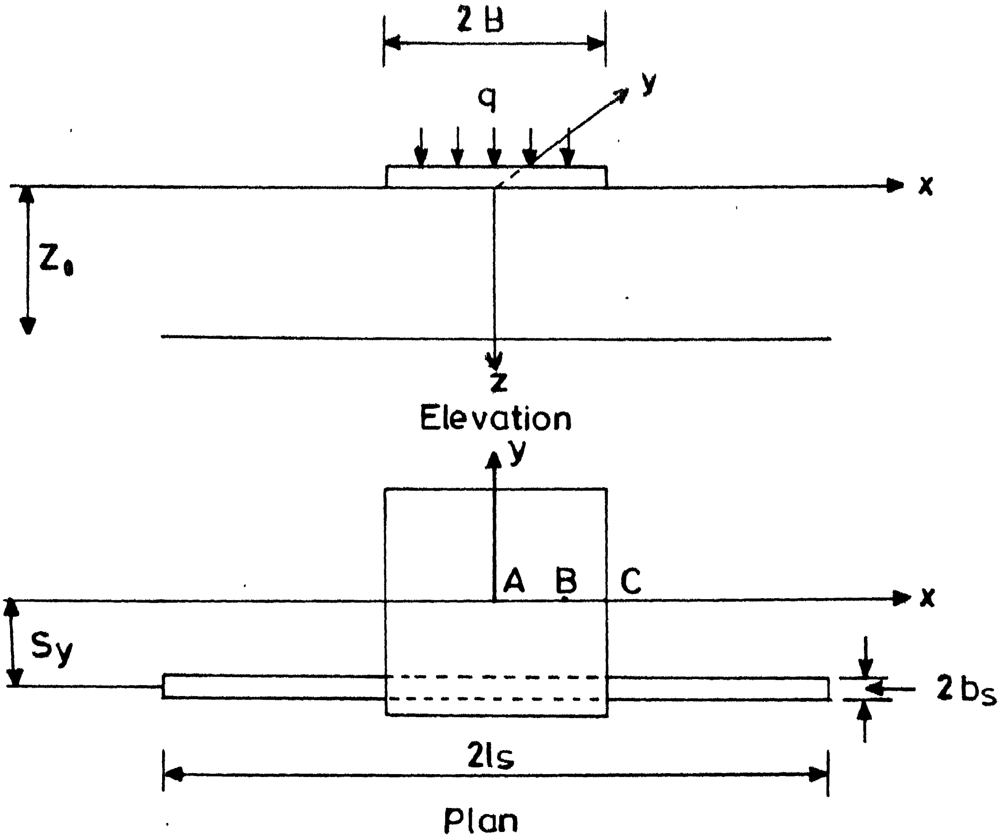

Fig. 3.1 Definition Sketch.

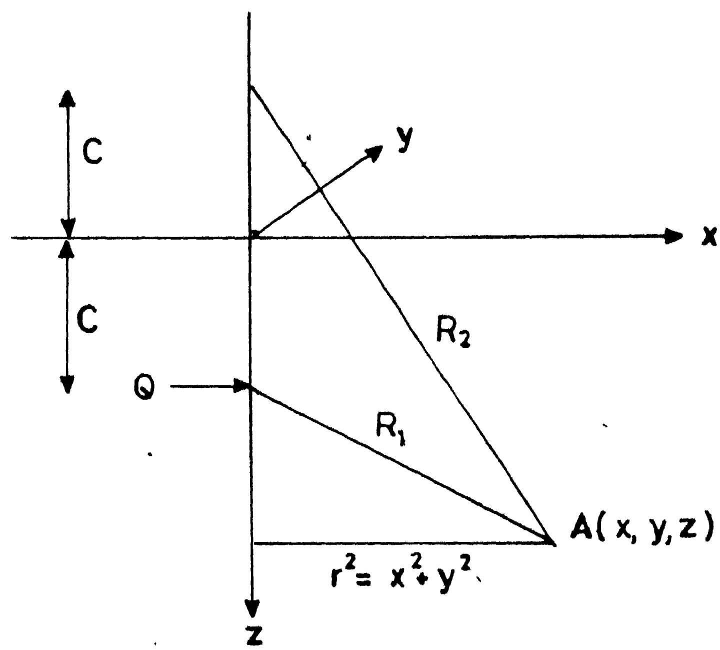

Fig. 3.2 Mindlin's Problem .

where $F = \frac{z-C}{R_1^3} + \frac{(3-4\nu).(z-C)}{R_2^3} - \frac{6.C.z.(z+C)}{R_2^5} + \frac{4.(1-\nu).(1-2\nu)}{R_2.(R_2+z+C)}$

$R_1^2 = x^2 + y^2 + (z-C)^2$

$R_2^2 = x^2 + y^2 + (z+C)^2$

ν = Poisson's Ratio

G = Shear Modulus of the soil

and x, y, z are the co-ordinates of point A relative to the force (Fig. 3.2).

In the present case, if the load Q in Eq. 3.1 is replaced by $\tau_{zx}.dA$, then the equation can be written as

$$d\rho_z = \frac{\tau_{zx}\ .\ dA}{16.\pi.G.(1-\nu)}\ .\ x\ .[\ F\ ] \qquad \ldots.(3.2)$$

i.e. the strip is discretized into small elements each of area dA. Eq. 3.2 is integrated along the length and width of the strip for the vertical movement of any point A due to the shear stresses along the strip

$$\text{i.e.} \quad \rho_z = \int_{-b_s}^{b_s} \int_{-l_s}^{l_s} d\rho_z$$

The vertical displacements at a point A(x, 0, 0) (Fig. 3.1), will reduce to

$$d\rho_z = \frac{\tau_{zx}(x).dx\ dy}{4\ \pi.G}\ .\ x\ .\ [\ D\ ] \qquad \ldots.(3.3)$$

where $D = [\ -\frac{z_0}{R^3} + \frac{(1-2\nu)}{R(R+z_0)}\ ]$

$R^2 = R_1^2 = R_2^2 = (x^2 + S_y^2 + z_0^2)$

$$\rho_z = \frac{1}{4.\pi.G} \cdot \int_{-b_s}^{b_s} \int_{-l_s}^{l_s} \tau_{zx}(x).D.x.dx.dy \qquad \ldots.(3.4)$$

Since the stresses acting along the strip are symmetrical about the centre and they act on both, the top and bottom, Eq. 3.4 gets modified as

$$\rho_z(0,0,0) = \frac{1}{\pi.G} \cdot \int_{-b_S}^{b_S} \int_{0}^{l_S} \tau_{zx}(x).D.x.dx\ dy \qquad \ldots.(3.5)$$

$$\rho_z(x_1,0,0) = \frac{1}{2.\pi.G} \cdot \int_{-b_S}^{b_S} \int_{-l_S}^{l_S} \tau_{zx}(x).D.x.dx.dy \qquad \ldots.(3.5a)$$

where x_1 is the distance of point B from the centre, and x, y, z are taken appropriately with reference to the point where the load is acting.

Eq.3.5 has been integrated numerically. The length of the strip is discretized into elements of length equal to 0.2B. Each element is further subdivided along the length, 0.2B and width, $2b_S$, into 32 subelements forming a grid. The stresses in all subelements within a particular element are assumed to be the same (Fig. 3.3). The influence of the shear stresses acting along the strip is evaluated at six equidistant points including the centre and the edge, along the footing half width, AC (Fig. 3.1). Convergence was checked by subdividing each element into 64 subelements.

Eq. 3.5a after numerical integration is written in local co-ordinates as

$$\frac{\rho_{ij}}{B} = \frac{1}{G} \cdot \tau_{zx}(x)_j \cdot I_{ij} \qquad \ldots.(3.6)$$

where ρ_{ij} is the displacement at point i due to stresses on element j

(Fig 3.3a)

I_{ij} is the influence of the stress acting on element j on element i.

$\tau_{zx}(x)_j$ is the shear stress on element j

I_{ij} is a non-dimensional term which can be written in local co-ordinates as

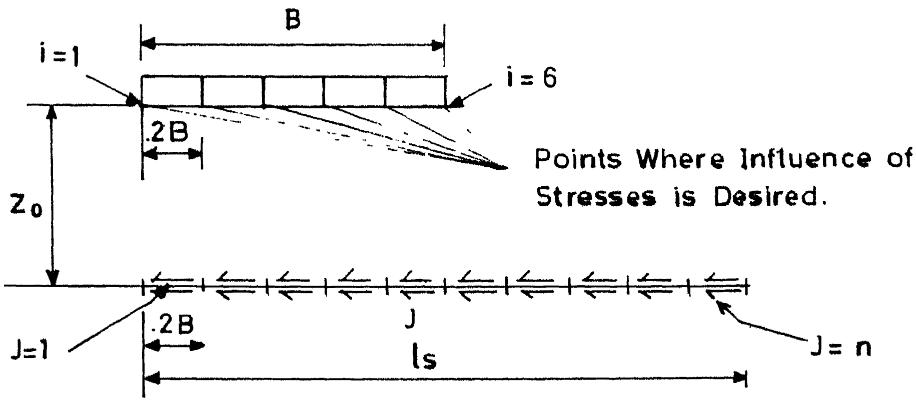

Fig. 3.3 (a) Discretization for Numerical Integration .

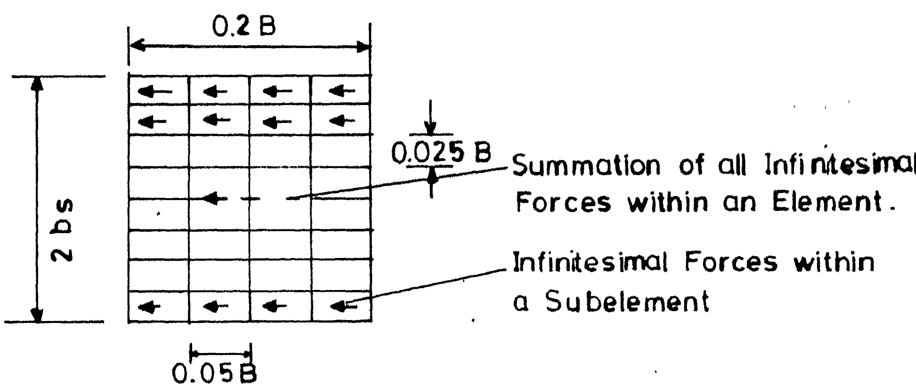

Fig. 3.3(b) Magnified View of an Element.

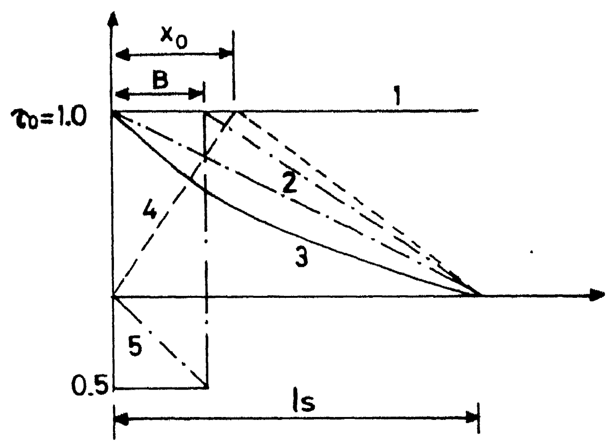

Fig. 3.4 Stress Distribution Along the Strip .

$$I_{ij} = \sum_{l=1}^{4} \sum_{p=1}^{8} \tau_{zx}(x)_j . D_1 . X . \Delta X \, \Delta Y$$

where l is an element along the length of the strip

p is an element along the width of the strip

D_1, X, ΔX, ΔY are all non-dimensional and the summation is carried out along b_s and over the length of an element j. The vertical displacement at any point can now be written as

$$\frac{\rho_i}{B} = \frac{1}{G} \cdot \tau_0 \cdot \sum_{j=1}^{n} I_{ij} \cdot f(x/B) \qquad \text{.... (3.7)}$$

In matrix form, Eq. 3.7 is written as

$$\{ \frac{\rho}{B} \} = \frac{\tau_0}{G} . [I] . \{ f(x/B) \} \qquad \text{.... (3 8)}$$

where [I] is an m X n matrix where m=6 is the number of points on the footing at which the influence is required.

n is the number of elements along the strip and

f(x/B) is the functional form of the variation of shear stresses along the length of the strip.

3.3 Stress Distribution along the Strip

Once the influence matrix I is obtained, the displacements of the points can be obtained knowing the distribution of stresses along the strip. Their exact distribution is not known but can be obtained by a detailed interaction analysis which is beyond the scope of this work Here five types of stress distributions are assumed (Fig. 3.4).

1 $\tau_{zx}(x) = \tau_0$; $0 \leq x/B \leq l_s/B$

2. $\tau_{zx}(x) = \tau_0 [\ 1 - \frac{x/B}{l_s/B}\ ]$; $0 \leq x/B \leq l_s/B$

3. $\tau_{zx}(x) = \tau_0 [\ 1 - (\frac{x/B}{l_s/B})^{n_1}\ ]$; $0 \leq x/B \leq l_s/B$; $n_1 < 1$

4. $\tau_{zx}(x) = \tau_0 . \frac{x/B}{x_0/B}$; $0 \leq x/B \leq x_0/B$

$\tau_{zx}(x) = \tau_0 [\ 1 - \frac{x/B}{l_s/B}\ ]$; $x_0/B < x/B \leq l_s/B$

5. $\tau_{zx}(x) = -\alpha . \tau_0 . x/B$; $0 \leq x/B \leq 1$

$\tau_{zx}(x) = \tau_0 [\ 1 - \frac{x/B}{l_s/B}\ ]$; $1 < x/B \leq l_s/B$

Type 4 distribution is the most realistic one In fact, this distribution is a linearised approximation of the stress distribution suggested by Binquet and Lee (1975). According to them the distance x_0, where τ_0 is a maximum is a function of the depth z_0. x_0 varies from 1B to 3.8B for z_0 varrying from 0.25B to 7B.

Type 5 distribution is an approximation of the stress distribution obtained from the analysis conducted by Brown and Poulos (1981). They have given the plot of the tension developed in the strips (Fig. 3.5). Differentiating these curves, the stresses in the strip can be obtained (Fig. 3.6). It can be seen that for layers lying closer to the footing, there exists a region extending from 0 to 0.8x/B along the length of the strip where the stresses are negative. Type 5 distribution takes these negative stresses into consideration.

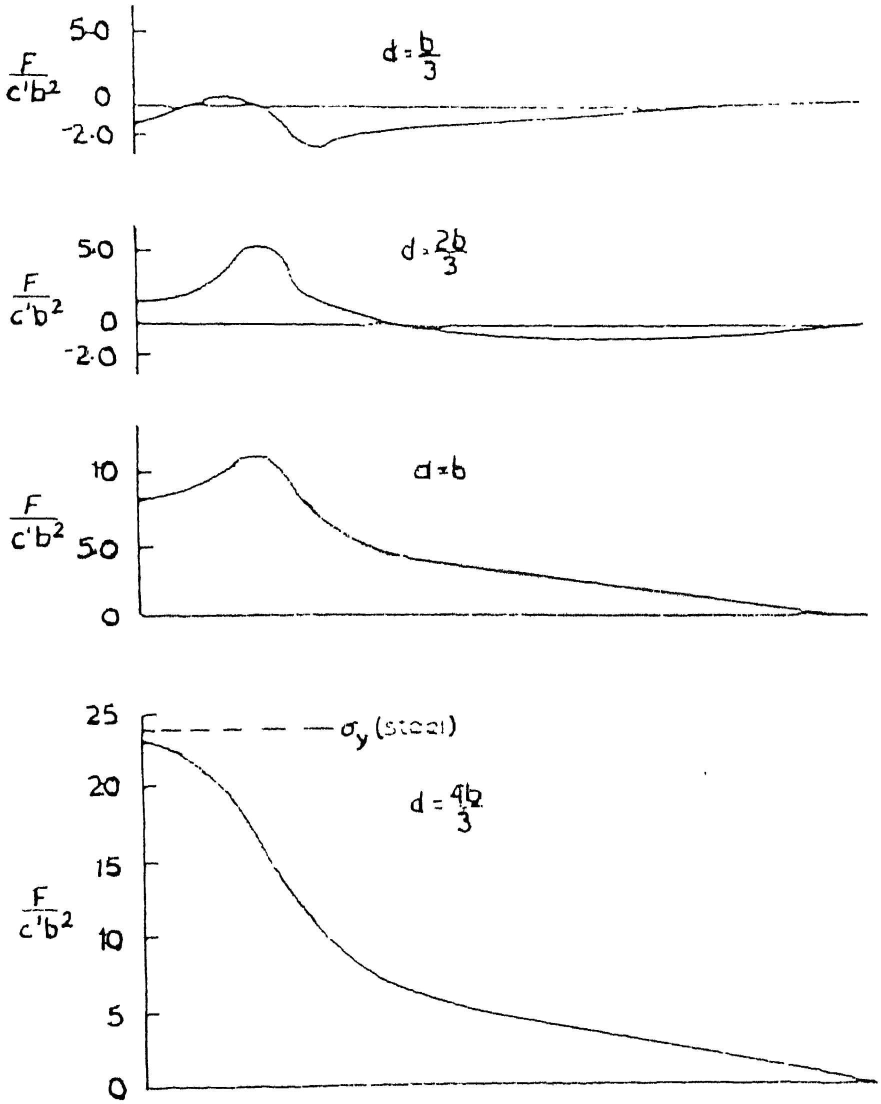

Fig. 3.5 Tension in the Reinforcements (Brown and Poulos)

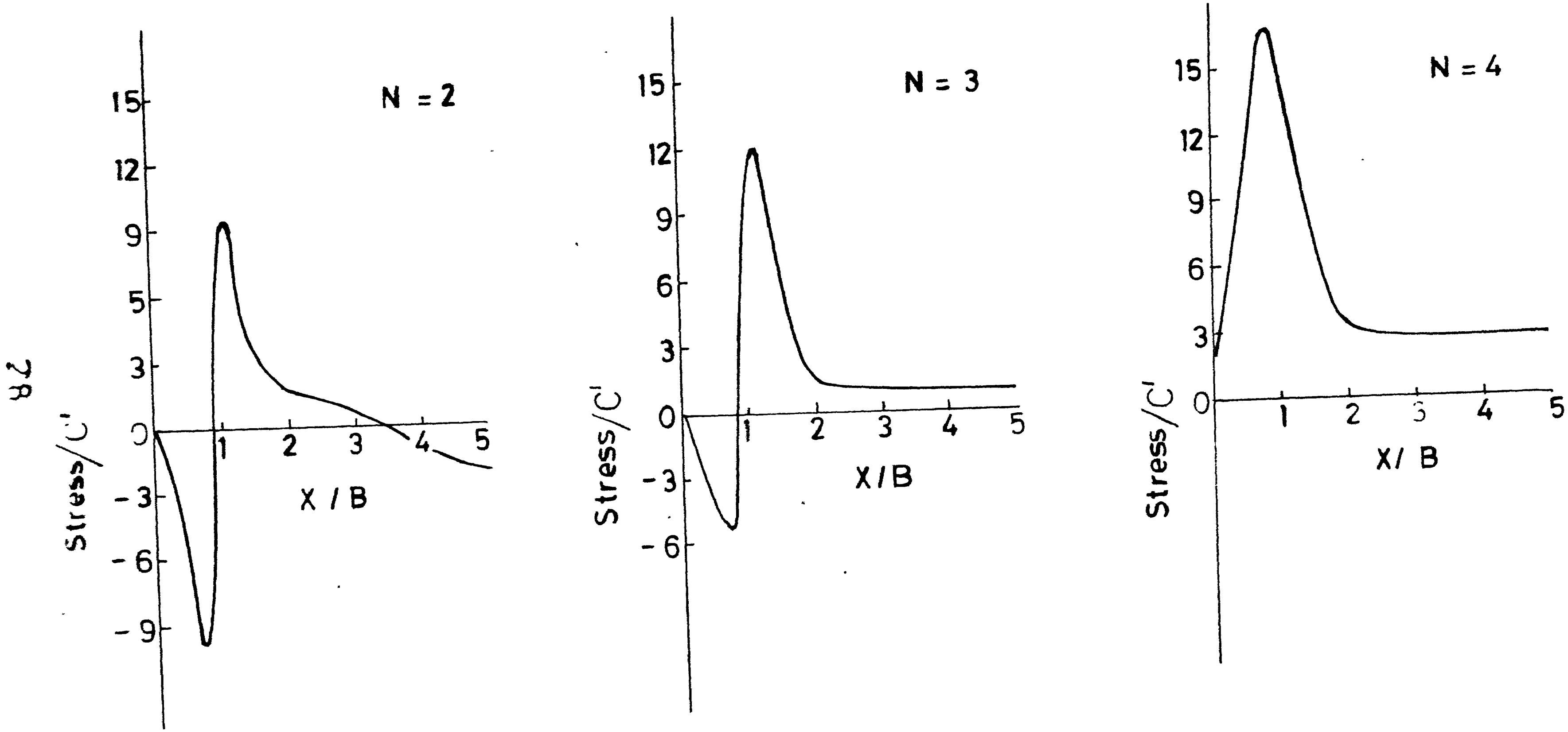

Fig. 3.6 Stress Distribution Along the Reinforcement (Brown and Poulos)

CHAPTER 4
RESULTS AND DISCUSSION

A computer program in Fortran IV was written to obtain settlement reduction coefficients (SRC), defined as $wG/B\tau_0$ and was run in the DEC-1090 computer system. Results were obtained for the following cases.

1. l_s/B = 2, 3, 4, 5
2. b_s/B = 0.1
3. S_y/B = 0, 0.25, ... 2
4. z_0/B = 0.25, 0.5, ... 2
5. 5 types of shear stress distributions.
6. Effect of Poisson's ratio, ν_s, in one case, while ν_s=0.3 for all other cases.

Positive values of SRC indicate heave of the surface while negative values of SRC mean that the soil at the surface is subjected to settlement.

The effect of the above mentioned parameters were studied on the settlement reduction at various points along the footing on the surface.

4.1 Effect of a Horizontal Force acting at a depth on Vertical Displacements at a point on the Surface

The basic effect of a horizontal force, Q, as shown in Fig. 4.1, acting at a depth, C, in a semi-infinite soil mass, on the vertical displacements of points on the surface has been studied .

The vertical displacement at a point A(x, 0, 0) is given by

$$w = \frac{Q.x}{4.\pi.G} . [- \frac{C}{R^3} + \frac{(1-2\nu_s)}{R(R+C)}] \qquad \ldots . (4.1)$$

$$= \frac{Q.C}{G} . I$$

where I is an influence coefficient dependent on x/C and ν_s.

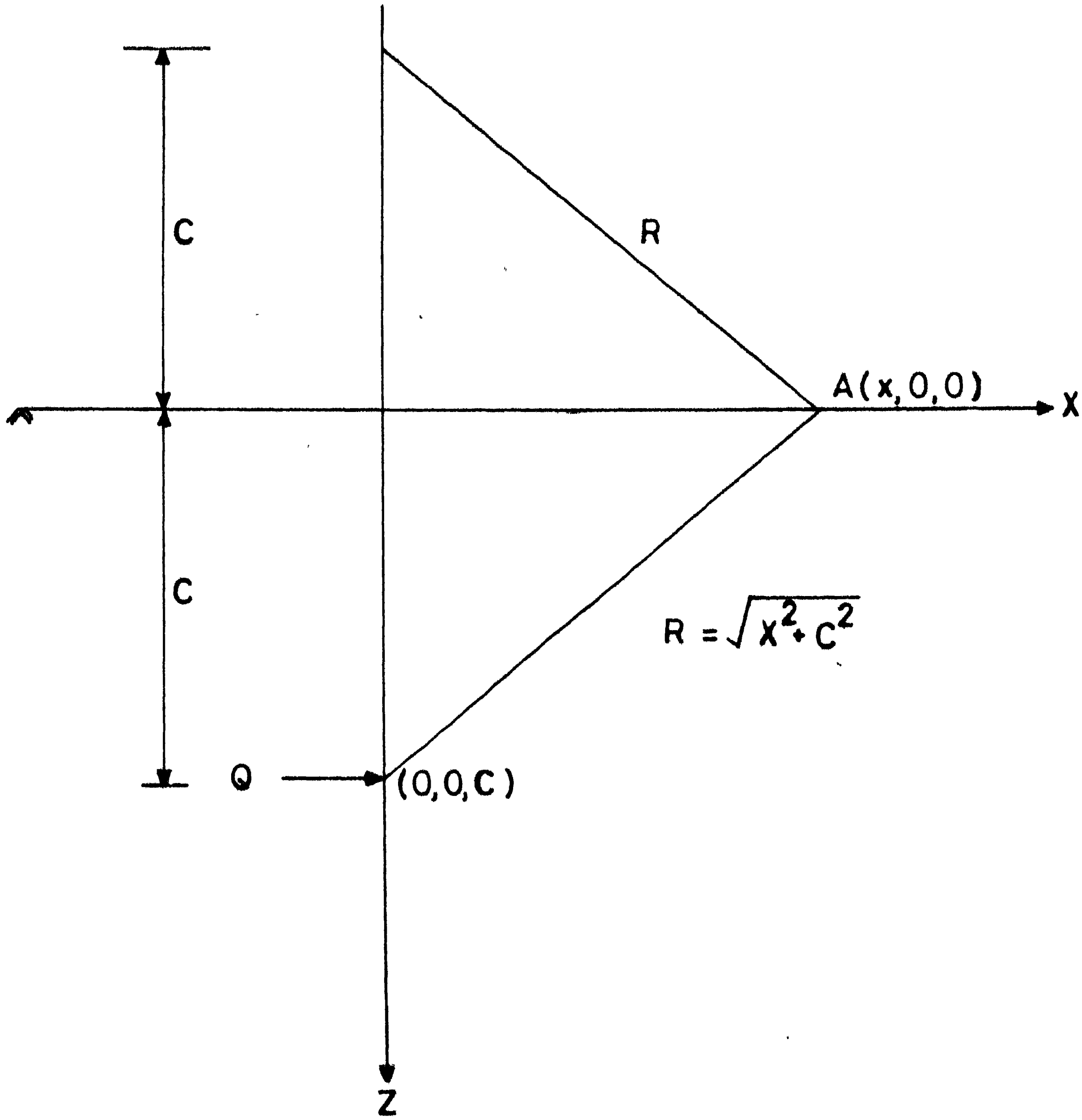

Fig. 4.1 Mindlin Problem: Horizontal Force Acting Below Ground.

Fig. 4.2 shows the plot of wG/QC versus x/C for different values of Poisson's ratio, ν_s. It can be seen that, for all the values of ν_s (except 0.5), there lies a region in the direction of the load where the soil moves upwards, ie., there is heave on the surface. Beyond this region the soil moves downwards and the curve assymtotically approaches the x-axis. Smaller the value of ν_s, smaller is this region and consequently, lower is the value of maximum heave. For all values of ν_s, the maximum heave occurs between 0.5x/C and 0.7x/C.

For ν_s=0.3, the maximum heave on the surface occurs at a distance of 0.65x/C from the force Q and the maximum value of w.G/Q.C is equal to 0.0224. At a distance of 3C from the force, the soil does not undergo any movement. Beyond this point the soil on the surface moves downwards, ie., settles with a maximum value of wG/QC equal to 0.002.

Saturated soils with ν_s = 0.5 show heave throughout the surface.

4.2 Effect of the Length of Strip

The plots of SRC at the centre of the footing versus l_s/B for the first four types of shear stress distribution are presented in Fig. 4.3. The graphs have been plotted for a depth ratio, z_0/B, of 0.5, distance ratio, S_y/B, of 0 and Poisson's ratio of 0.3.

For type 1 shear stress distribution, the SRC at the centre of the footing increases from l_s/B = 1 and attains a maximum value of about 0.0244 for l_s/B = 1.5 and then decreases rapidly. For l_s/B = 10, the SRC is practically zero.

In the case of types 2 and 3 distributions, the maximum values of SRC ar observed for the length ratio of the strip, l_s/B = 3 and 4 repectively. SR decreases gradually with further increase in l_s/B. At l_s/B = 10, SRC of 0.01

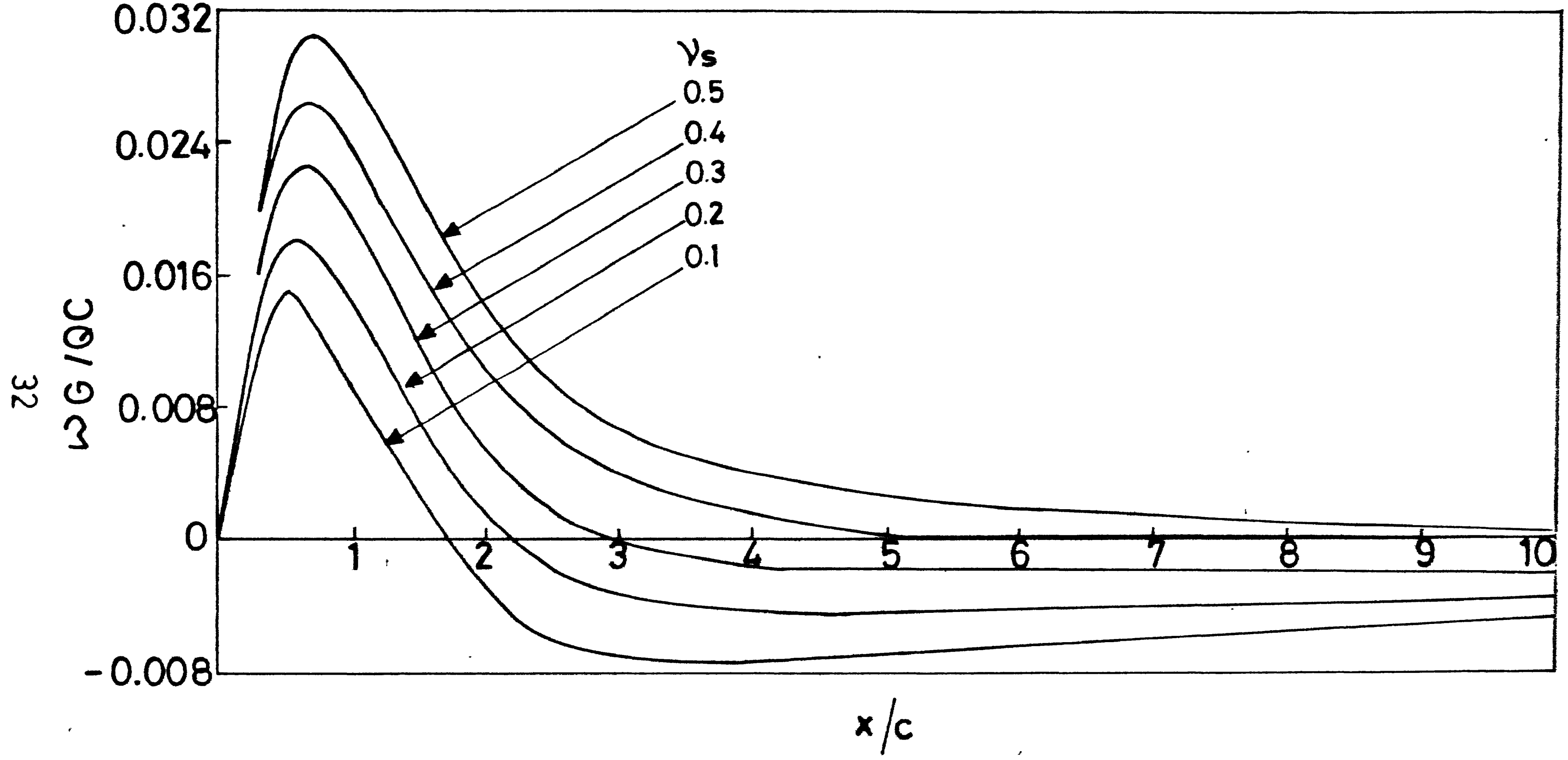

Fig. 4.2 Variation of wG/QC with x/c for Different Values of Poisson's Ratio, ν_s.

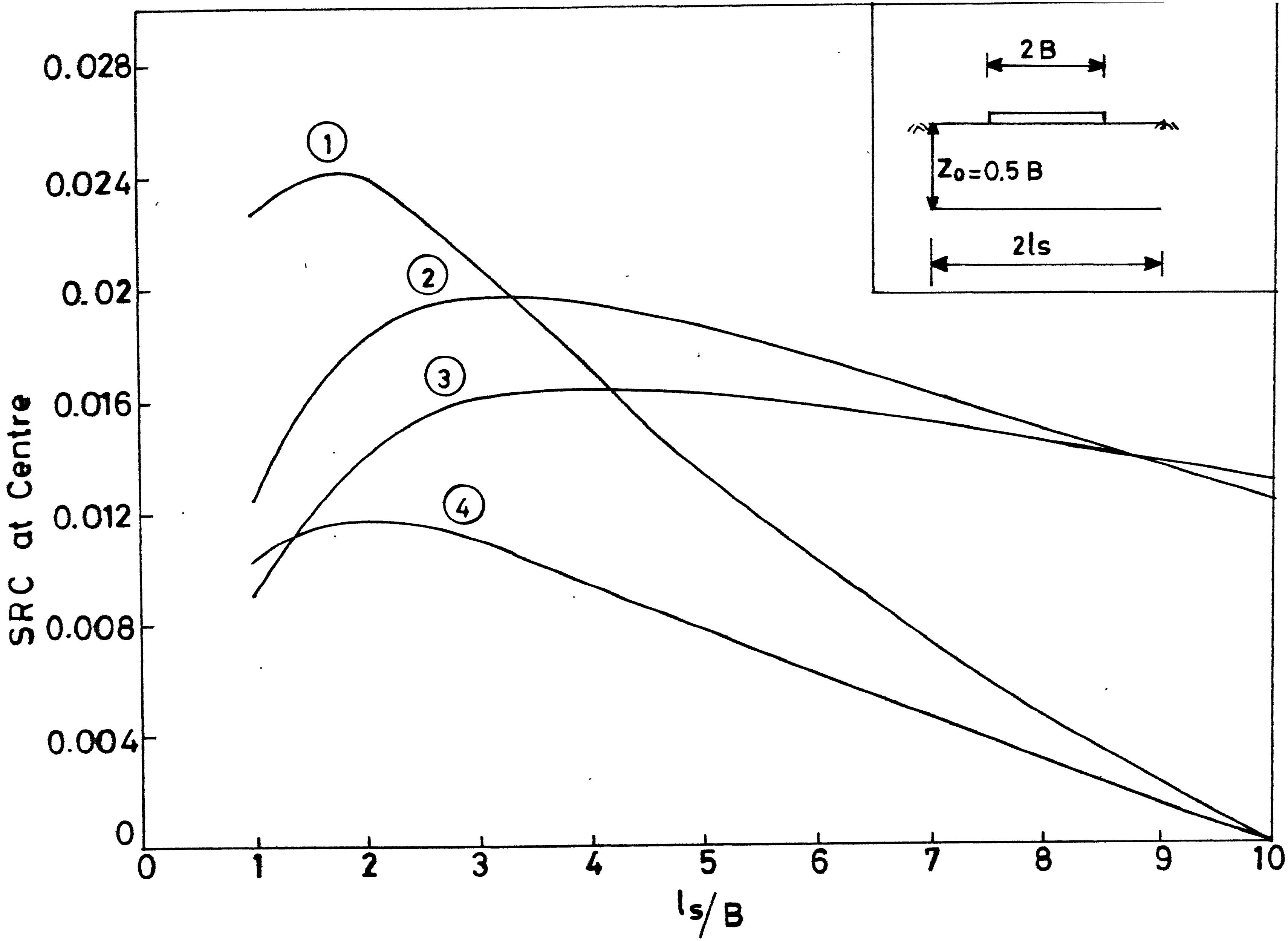

Fig.4.3 Variation of SRC at the Cenre of Footing with l_s/B for the Different Types of Shear Stress Distribution.

and 0.0132 are observed for types 2 and 3 distribution respectively.

Type 4 distribution shows a maximum value of SRC of 0.0118 for $l_s/B = 2$ and decreases continuously with further increasse in l_s/B, becoming almost zero at $l_s/B = 10$.

Fig. 4.4 shows the variation of SRC at the centre of the footing and at its edge with the length ratio of the strip, l_s/B for type 5 stress distribution. These curves have been plotted for a depth ratio, z_0/B of 0.5 and for the strip lying concentrically below the footing. It can be seen from the figure that there is heave at the edge of the footing while the centre undergoes settlement. Maximum heave is observed for a length of the strip, $l_s = 3B$. As the length of the strip increases, the heave at the edge of the footing decreases while settlement at the centre of the footing increases.

It can be seen from the above results that any increase in the length of the reinforcing strip beyond an optimum length, for any type of shear stress distribution is not effective. An explanation of this behaviour can be given on the basis of the discussion in art. 4.1 and with reference to Fig 4.2. If the stresses along the strip fall beyond the region where heave of the soil is created, and within the zone of settlement, there will be downward movement of the soil on the surface, thus reducing the heave. In the case of type 5 distribution, the negative shear stresses within a length of 1B along the strip cause the soil at the centre of the footing to settle while the soil at the edge is pushed up.

4.3 Effect of Location of the Strip

Fig. 4.5 shows the variation of SRC at the centre of the footing with the distance of the strip, S_r/B, from the axis of symmetry for various values of the depth of placement, z_0/B and length of the strip, l_s/B. It is evident from these

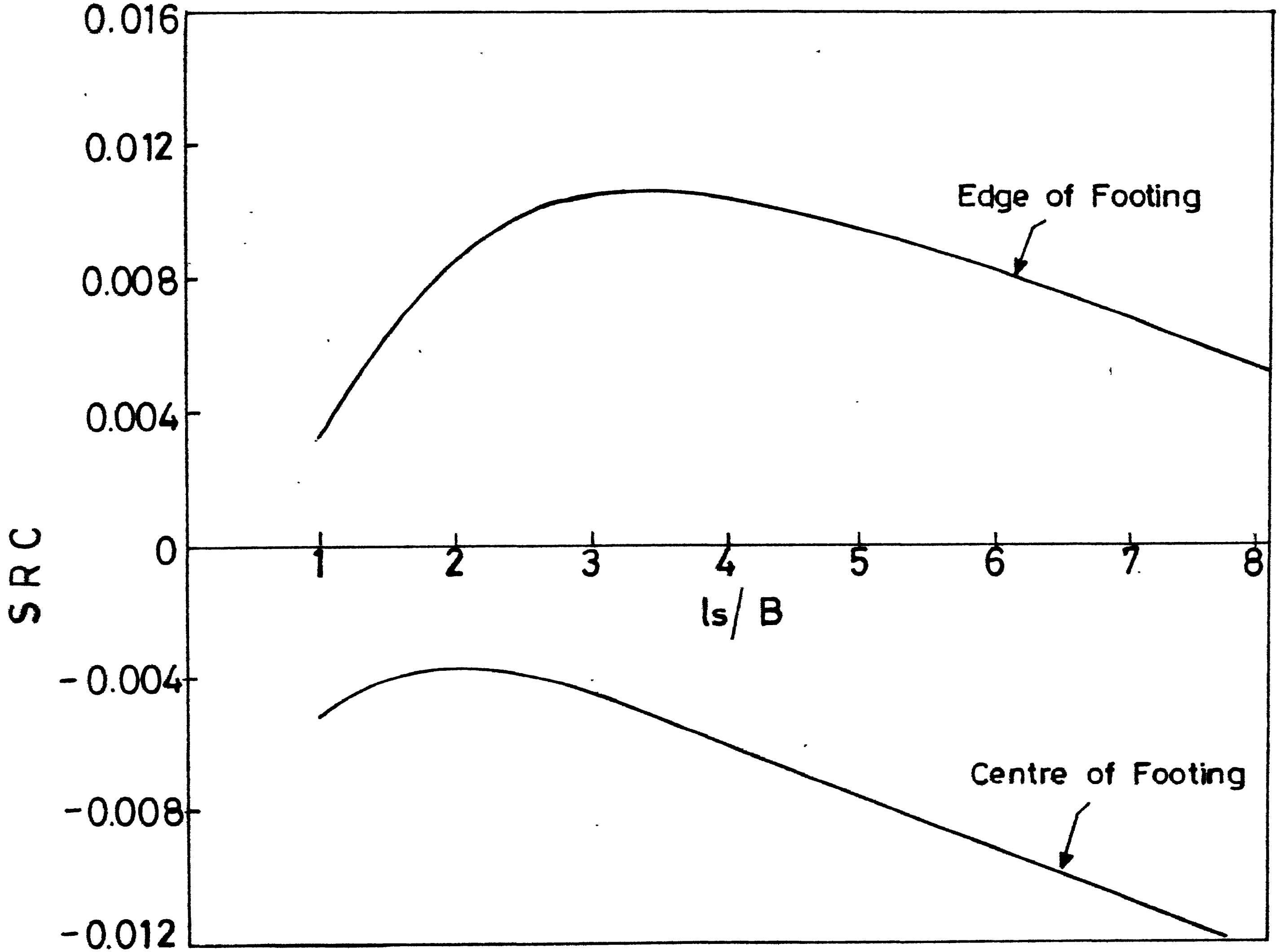

Fig. 4.4 Variation of SRC with ls/B (Type 5 Distribution, $Z_\phi/B=0.5$, $Sy/B=0$)

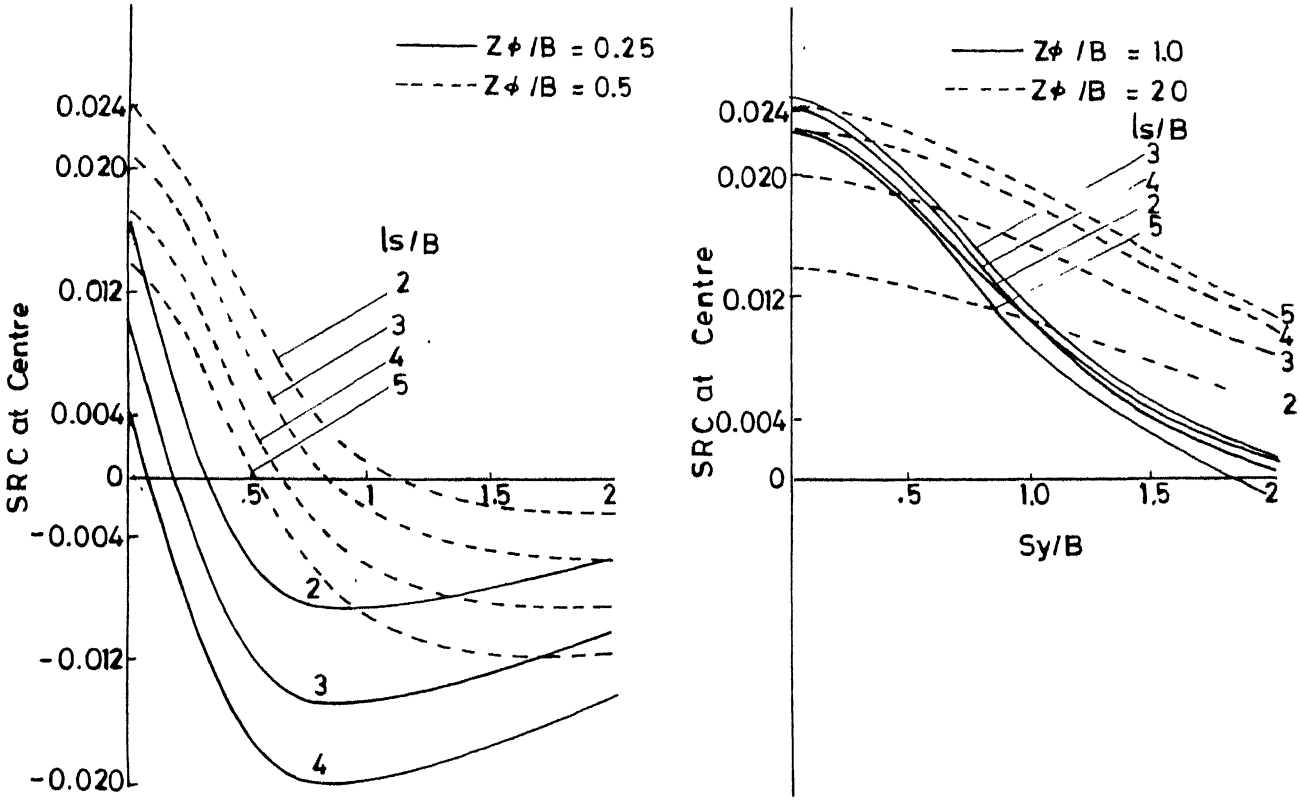

Fig 4.5(a) Type 1 Stress Distribution

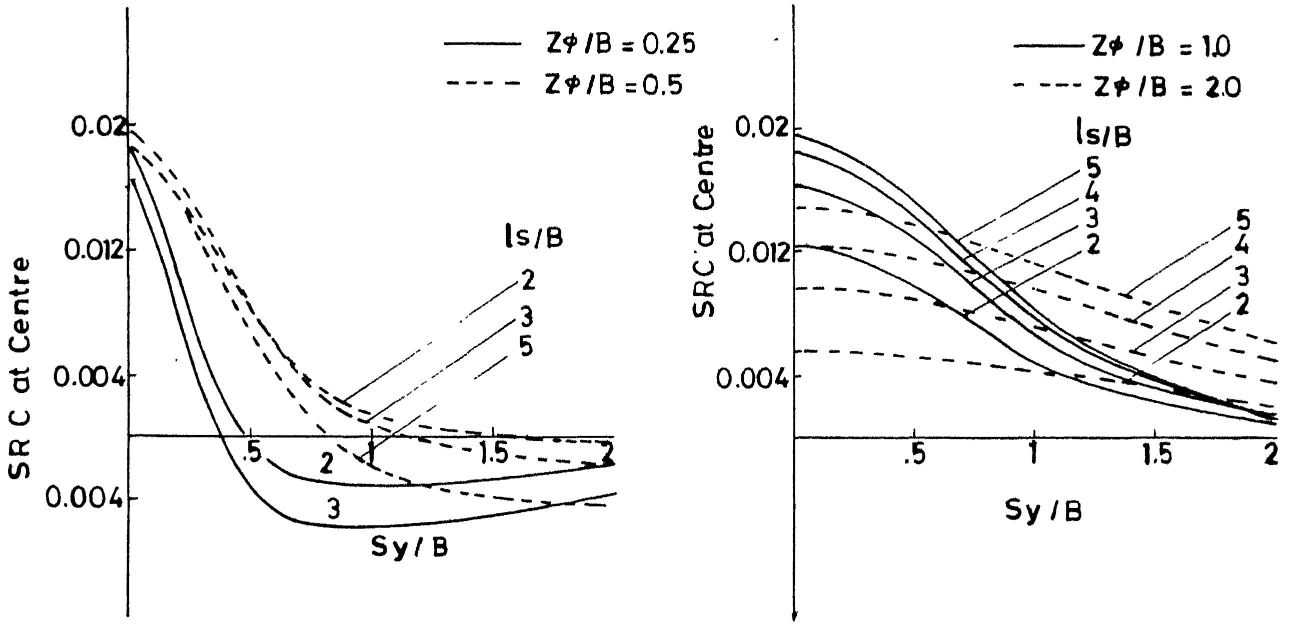

Fig. 45 (b) Type 2 Stress Distribution

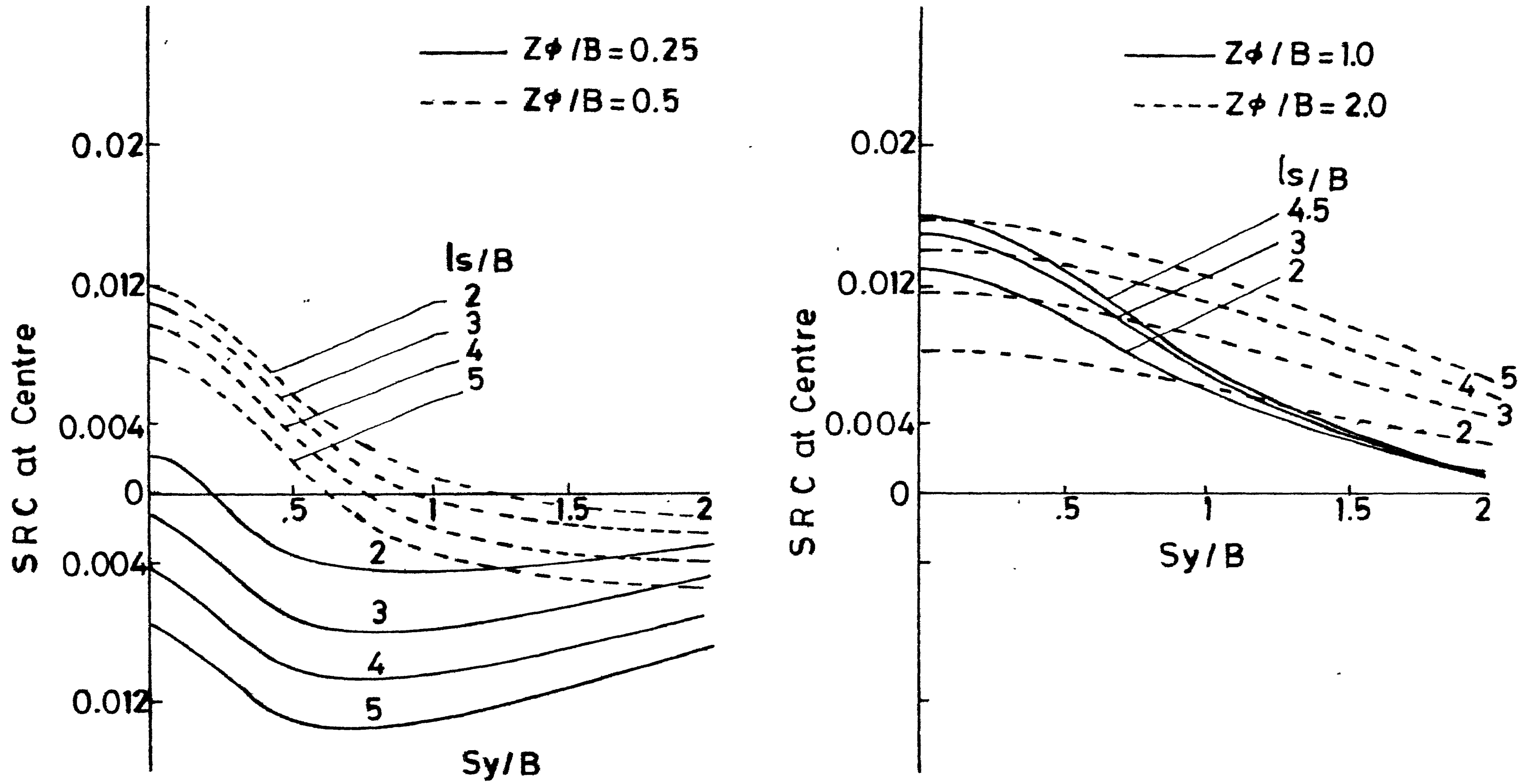

Fig.4.5(c) Type 4 Stress Distribution

Fig. 4.5 Variation of SRC at Centre of Footing with Sy/B for Different Values of Zϕ/B and ls/B.

graphs that for any type of stress distribution the SRC decreases with increase in the distance of the strip from the axis.

For type 1 distribution, when the strip is placed at a depth of 0.25B and concentrically below the footing, ie., $S_r/B = 0$, there is heave on the surface. With further increase in the distance of the strip from the axis, the SRC reduces and becomes negative, ie., the point settles instead of heaving. When S_r/B is 0.75, the settlement on the surface is maximum and any further increasse in S_r/B reduces the downward displacement of the soil. This clearly indicates that at larger distances of placement of the strip from the point under consideration, the effect of the strip on the settlement reduction on the surface becomes negligible. For $z_0/B = 0.5$ the soil settles for S_r/B greater than that for $z_0 = 0.25B$. As the depth of placement of the strip increases the curve becomes less steeper and at a depth of $z_0 = 2B$, there is heave for distances (S_r) as large as 2.

One more aspect worth noting is that, the SRC is dependent on the length of the strip also. As can be seen, there is a reversal in trend as the depth increases. For eg., at depths of placement of the strips of 0.25B and 0.5B, strips with $l_s/B = 2$ give larger values of SRC as compared to strips with $l_s/B > 2$. At a depth of $z_0 = B$, there is a reversal in the trend and at a depth of $z_0 = 2B$, strips with $l_s/B = 5$ show higher SRC than strips with $l_s/B < 5$.

Stress distributions of types 2, 3 and 4 also show similar trends. In type 2 distribution, the reversal of trend is observed at a depth, $z_0 = 0.5B$ while for type 4 distribution this is observed at a depth, $z_0 = B$. It is interesting to note that in this case (type 4 distribution) SRC is nearly the same for $S_r/B \geq 1$ for $z_0/B = 1$ for all lengths of the strip, which again means there is no advantage in providing strips longer than an optimum length.

Fig 4.6 shows the variation of SRC at the centre and edge of the footing with z_0/B for different values of S_y/B and l_s/B for type 5 stress distribution. From this figure it can be seen that for $l_s/B = 2$ there appears to be more settlement at the centre of the footing when the depth of placement is 0.25B as compared to 0.1B, for $S_y/B = 0$. As the length increases, the settlement for smaller values of z_0/B increases and SRC for larger depths of placement increases. The improvement in SRC for $l_s/B = 5$ as compared to $l_s/B = 4$ is not significant, indicating that, providing lengths of strips longer than an optimum length is not advantageous. It can also be seen that as S_y/B increases, the SRC decreases.

Fig 4.7 shows the plot of SRC versus the distance along the width of the footing, x/B for different values of S_y/B, z_0/B and l_s/B for type 5 distribution. It is evident from these curves that, for smaller values of S_y/B, there is considerable variation in the SRC along the width of the footing. For $S_y/B = 1$, SRC along the width of the footing becomes almost constant. This again indicates that as the distance of the strip from the centre increases, the effect of the strip on SRC decreases. Further, for $z_0/B = 2$, the SRC is almost constant even when the strip is placed concentrically below the footing.

4.4 Effect of Depth of Placement of the Strip

The depth at which the strip is placed also affects the settlement reduction. This effect is brought to light in the figures 4.8, 4.9, and 4.10. The plot of SRC at the centre versus z_0/B, for various values of S_y/B and l_s/B is shown in these figures. For all types of stress distributions it is noticed that the SRC increases initially with an increase in z_0/B, reaches a maximum value and then decreases. There is a value of z_0/B, for a given length of the strip and distance, S_y, for which the SRC is maximum. For a particular type of stress distribution, the

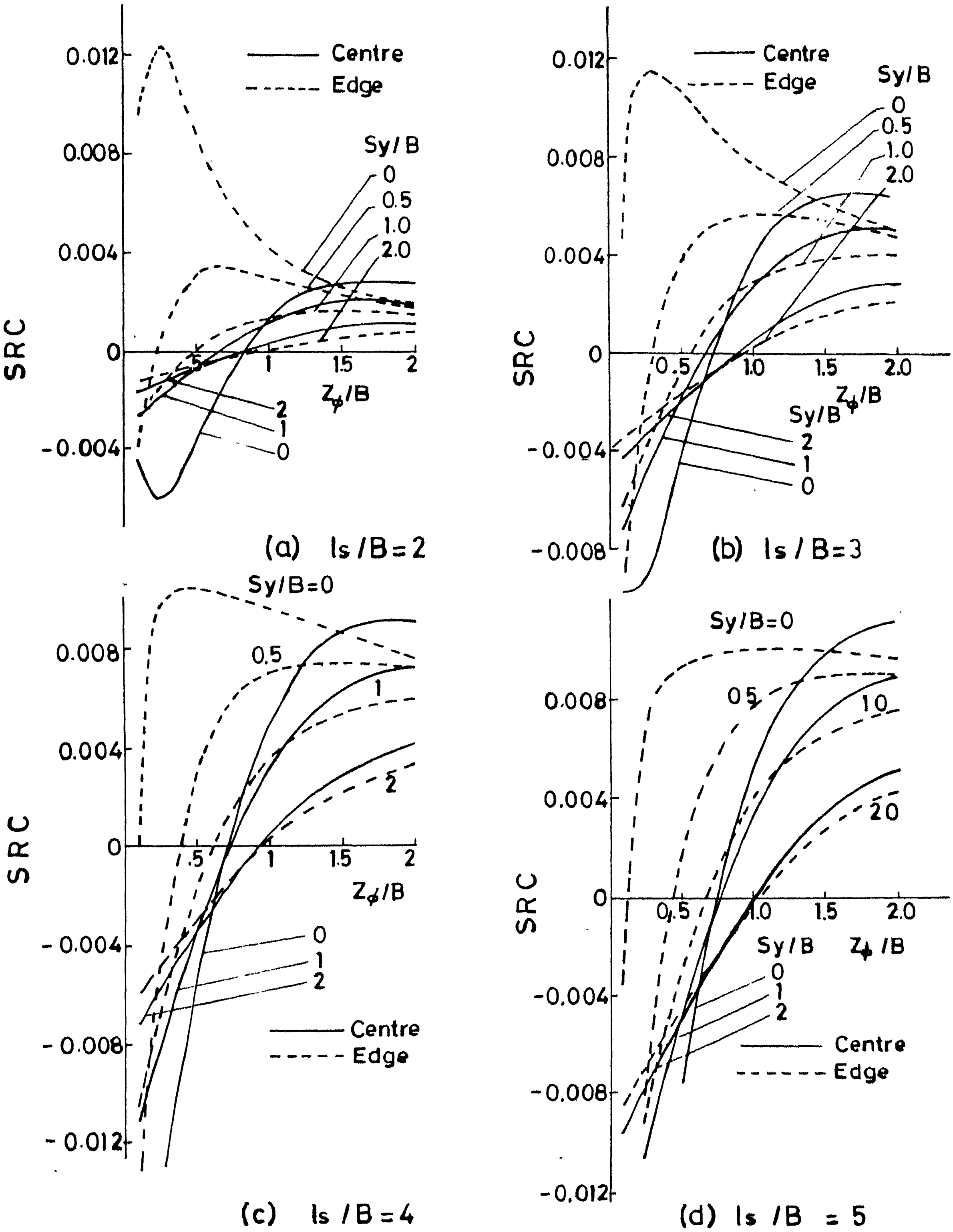

Fig. 4.6 Variation of SRC with Z_ϕ/B for Different Values of Sy/B and ls/B at Centre and Edge of Footing (Type 5)

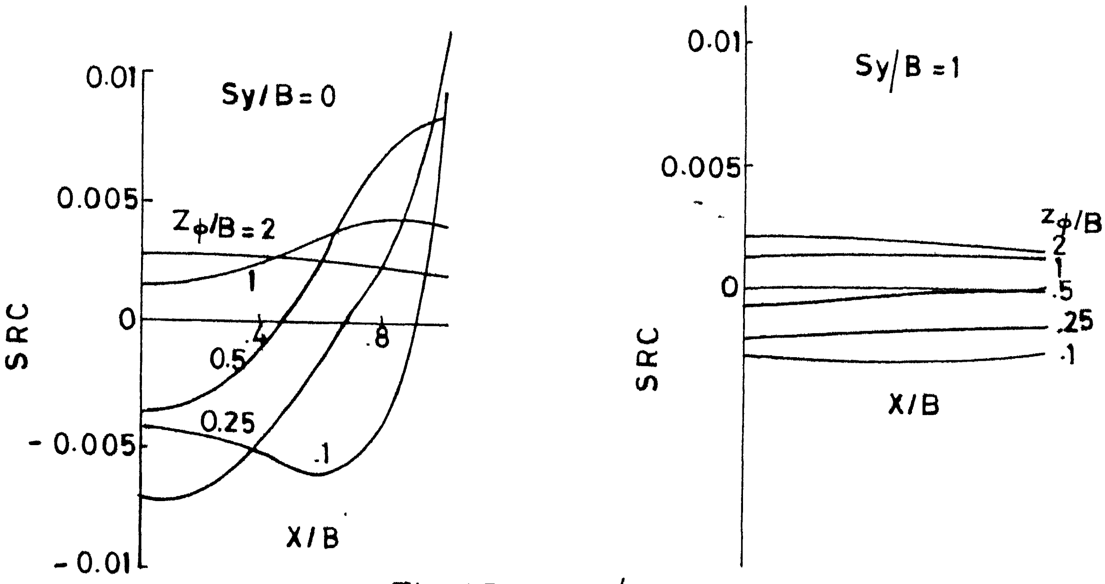

Fig. 4.7 (a) ls/B = 2

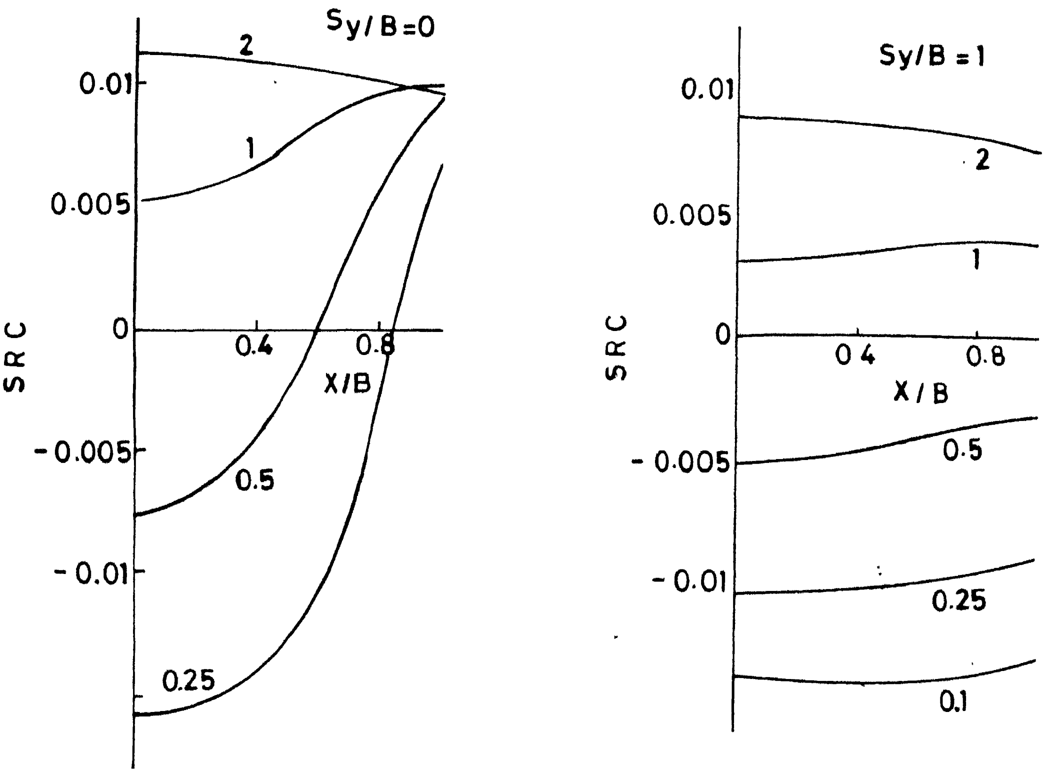

Fig. 4.7 (b) ls / B = 5

Fig. 4.7 Variation of SRC with x/B for Different Sy/B and Z_{ϕ}/B (Type 5)

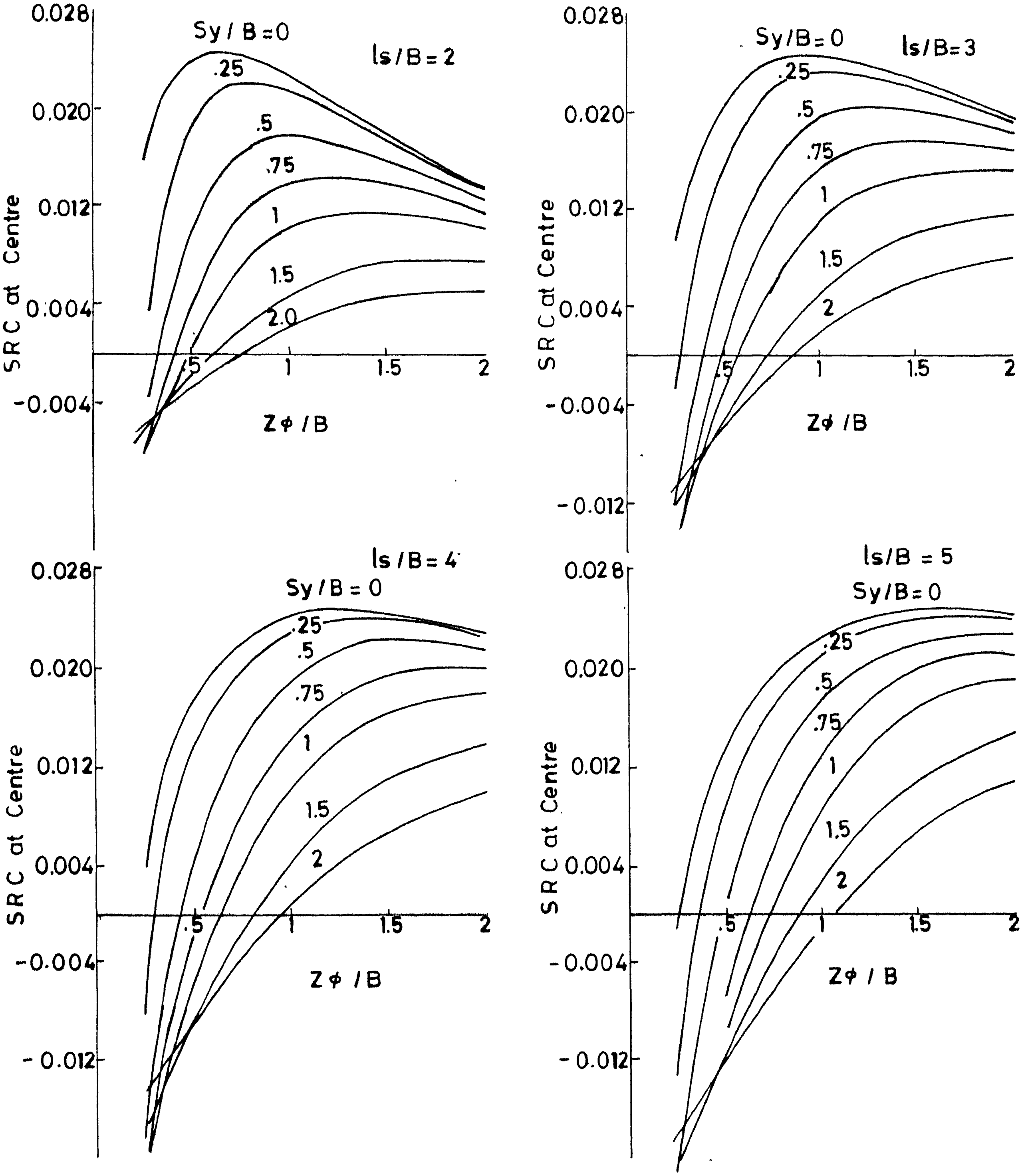

Fig. 4.8 Variation of SRC at Centre of Footing with Zϕ/B for Different Values of Sy/B and ls/B (Type 1)

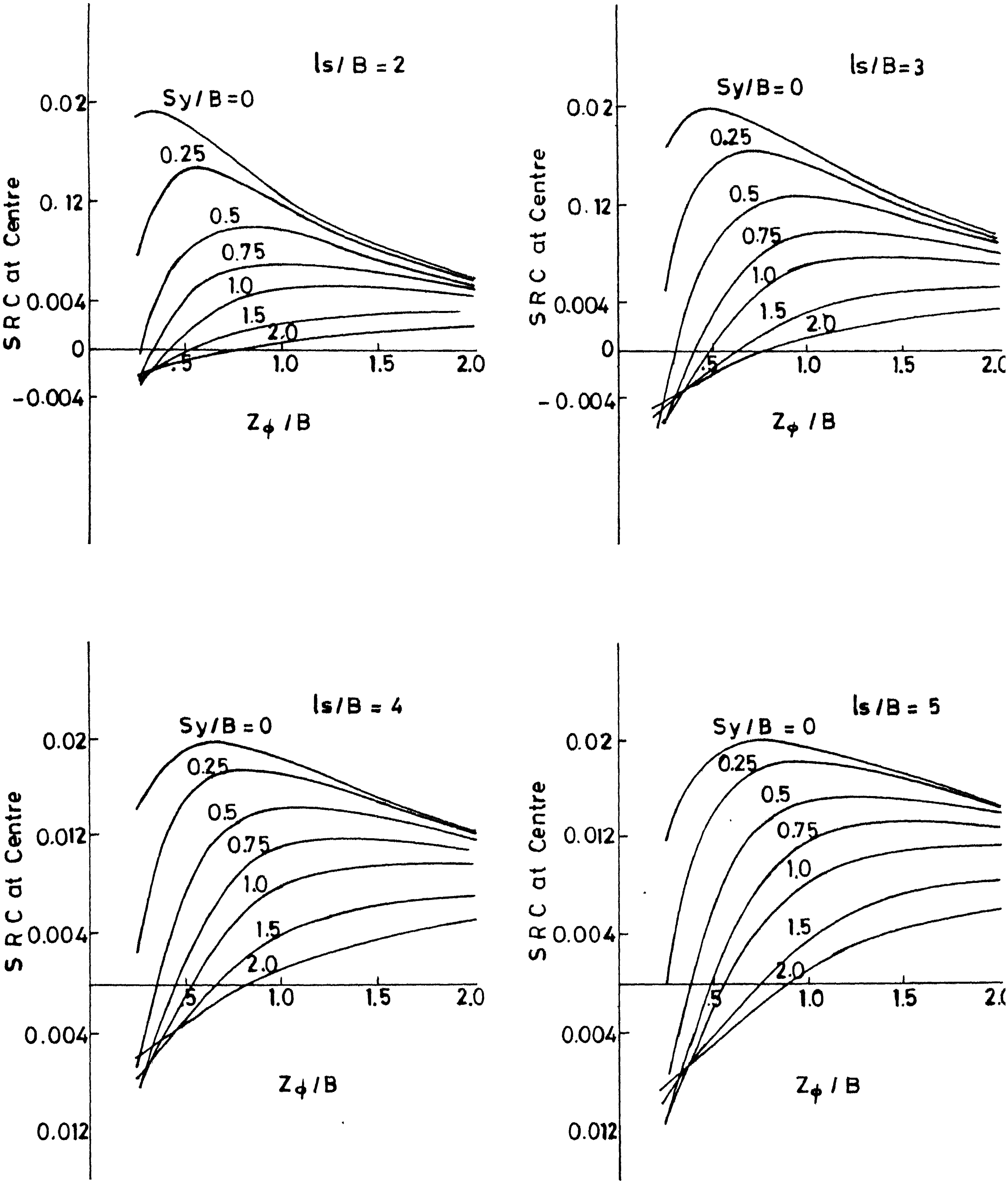

Fig. 4.9 Variation of SRC at Centre of Footing with Z_ϕ/B for Different Values of Sy/B and ls/B (Type 2)

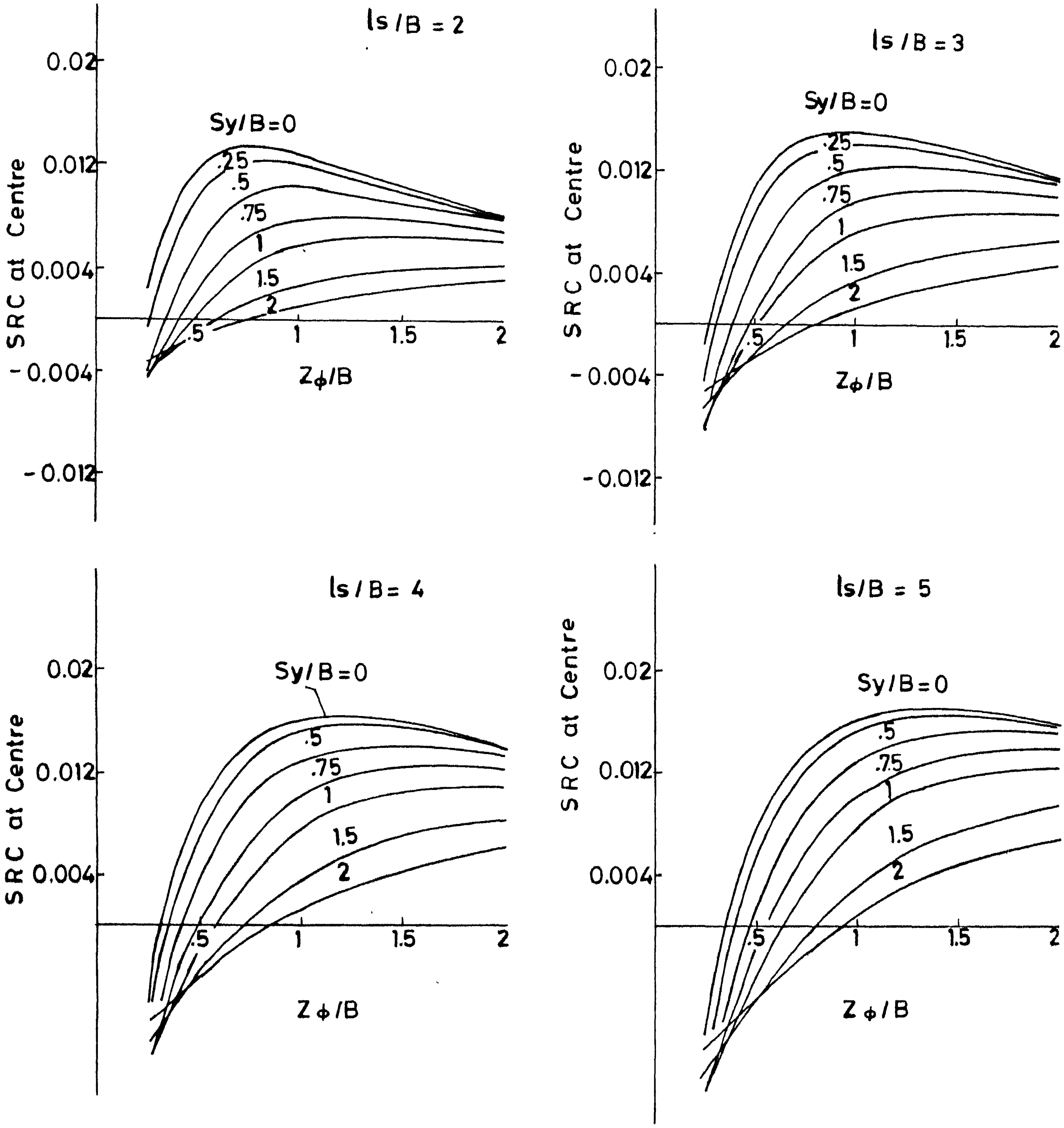

Fig. 4.10 Variation of SRC at Centre of Footing with Z_ϕ/B for Different Values of Sy/B and ls/B (Type 4)

maximum value of SRC depends on the length of the strip and the depth of its placement.

For all types of stress distribution, as the length of the strip increases, the SRC for a depth of 0.25B always decreases while for a depth of 1.5B it always increases. It is evident from this behaviour that the effect of longer strips on SRC is enhanced at greater depths.

For type 1 distribution, the maximum SRC for $S_y/B = 0$ is observed at around $z_0/B = 0.6$ for a length of the strip, $l_s = 2B$ whereas for $l_s/B = 5$, the maximum occurs at $z_0/B = 1.5$ For the length of the strip, $l_s = 2B$ and depth of placement, $z_0 = 2B$, the SRC ranges from 0.014 to 0.005 for S_y/B ranging from 0 to 2. For the length of the strip, $l_s = 5B$, under the same conditions as above, the SRC ranges from 0.0244 to 0.0108. This shows that the effect of the distance of placement of the strip, S_y on SRC is less in case of shorter strips as compared to longer strips at the same depths of placement.

For type 2 distribution, the maximum SRC for $S_y/B = 0$ is seen at $z_o/B = 0.3$ for the length of the strip, $l_s = 2B$, while for a strip with $l_s/B = 5$, the maximum SRC is observed at a depth, $z_0 = 0.75B$. The convergence here is faster than that for type 1 distribution. Curves for type 3 distribution are very much similar to those of type 2 and thus are not presented.

Type 4 distribution also shows similar trends. As long as the strip of any length is placed at a depth greater than 0.75B, there will be settlement reduction on the surface. For type 5 distribution (Fig. 4.6) it can be seen that as the depth of placement of the strip increases, the difference in SRC at the centre and edge of the footing decreases. For depths of placement, $z_0 \geq B$ there will always be heave on the surface. One more fact that is worth noting is that at very shallow

depths of placement, the SRC at the edge of the footing is always greater than that at the centre. As the depth increases, the SRC at the centre of the footing starts improving and at a depth of 2B, it is greater than that at the edge of the footing.

Fig. 4.11 shows the variation of z_{0max}/B versus S_y/B for different lengths of the strip. It can be seen that, as S_y/B increases, the depth for maximum SRC also increases. Further, as the length of the strip increases, the value of z_{0max} increases for a particular value of S_y/B.

4.5 Effect of Poisson's Ratio on SRC

The effect of poisson's ratio on SRC is shown in Fig. 4.12. Unlike the other graphs, here the SRC is calculated for the elements along the footing as shown in Fig. 3.3a. These elements include the centre of the footing, its edge and four points along the width of the footing at an interval of 0.2B. Except for the central point on the footing, all other points where the SRC is calculated are not symmetric with respect to the length of the strip.

Curves have been drawn for SRC versus the distance along the width of the footing, for various values of ν_s. The distance of the strip from the centre of the footing is 0.5B, the depth of placement, z_0 is 0.5B and the length of the strip is 3B. Curves have been drawn for the first four types of stress distribution. For all the types of distributions, it can be noticed that the soils with poisson's ratio ranging from 0 to 0.2 settle. Even at the centre of the footing there is hardly any settlement reduction. Saturated soils show heave throughout the surface.

One more fact worth noting is that, as the ν_s increases there is a change in the curvature of the curves. The graphs seem to converge at some distant point from the centre of the footing. This indicates that the effect of ν_s on the SRC

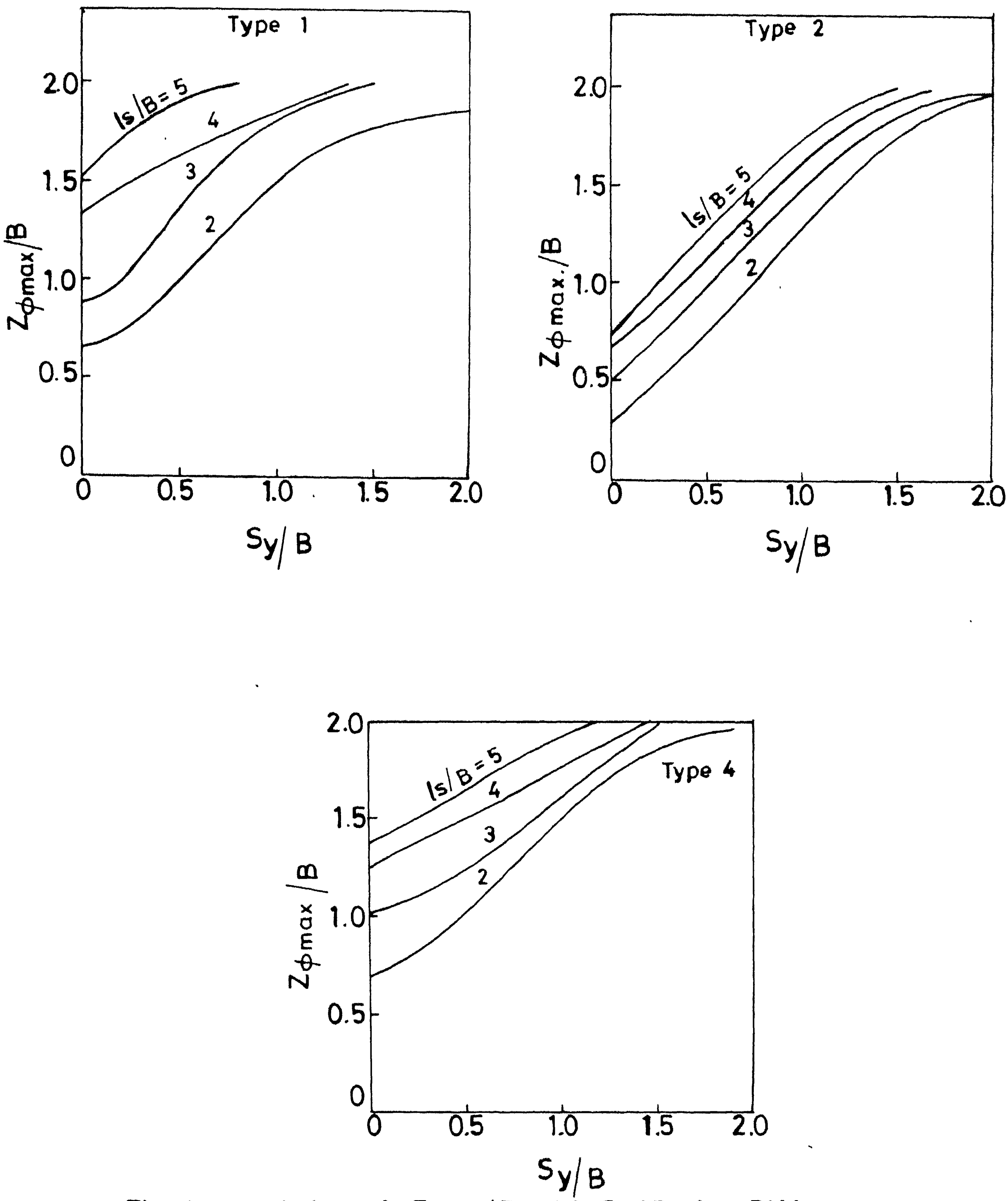

Fig. 4.11 Variation of $Z_{\phi max}/B$ with Sy/B for Different Values ls/B and Types of Stress Distribution.

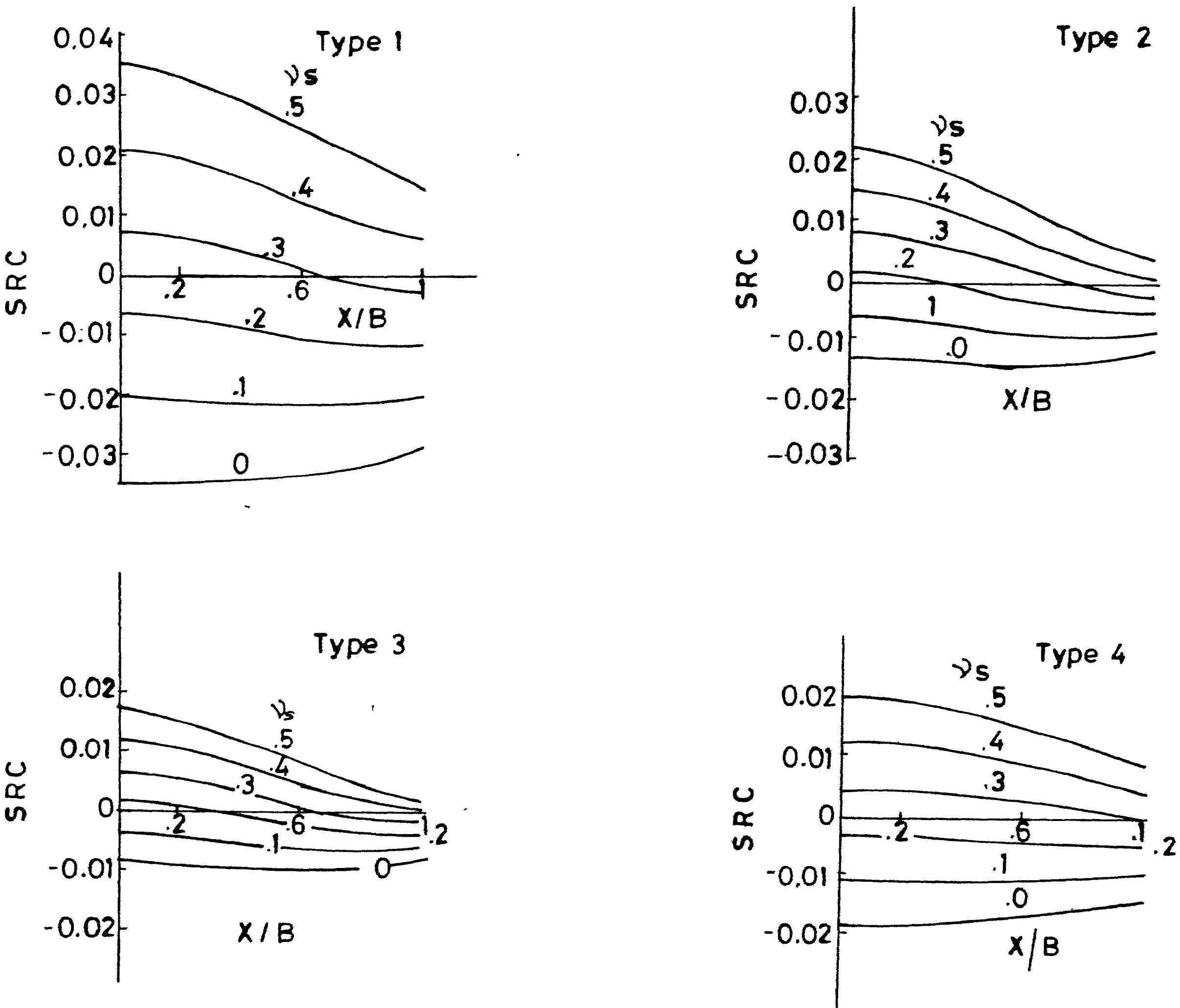

Fig. 4.12 Variation of SRC with X/B for Different Values of Poisson's Ratio, ν_s and Types of Stress Distribution ($l_s/B = 3$, $S_y/B = 0.5$ and $Z_\phi/B = 0.5$)

becomes negligible at greater distances along the x-direction.

Type 1 distribution shows a larger variation in SRC with the variation in ν_s as compared to all other types of stress distribution. For this type the SRC at the centre, for $\nu_s = 0$, is -0.035 (settlement) whereas for $\nu_s = 0.5$, it is 0.035 (heave). For type 2 distribution these values are -0.014 (settlement) and 0.022 (heave), while type 4 distribution indicates values of -0.018 (setttlement) and 0.02 (heave). Type 3 distribution shows a minimum variation of -0.0085 (settlement) and 0.017 (heave). These studies show that for soils with smaller poisson's ratio, less effective are the shear stresses in reducing settlements.

It can be visualised from the above sections that the effect of each parameter ie. length, distance, and depth of placement of the strip cannot be considered separately. Each parameter has his own effect on the other. Hence a concurrent consideration of all these parameters reveal that for obtaining the maximum SRC, the strip should be placed suitably considering all the three aspects.

Smaller the distance S_y, greater is the effect of the strip on SRC. Once the distance S_y is finalised, the depth of placement must be chosen based on the length of the strip or vice versa.

To obtain the effect of a number of strips, the SRC due to each strip is considered independently and summed up, ie. the validity of the principle of superposition is assumed.

4.6 Comparison with available results

4.6.1 *Qualitative Comparison*

Brown and Poulos (1981) commented that the reinforcement will be more effective at greater depths. They used strips of length 5B. They concluded that

strips placed at a depth of 1.3B are more effective than those placed at a depth of B or 0.66B. Considering Fig. 4.10 it can be seen that there is good agreement qualitatively with this fact for $l_r/B = 5$ and type 4 distribution.

Patel (1988) stated from his model studies that beyond a depth of 3.3B the effect of the geotextile is negligible. This fact is also clearly noticeable from the trends shown in Fig. 4.10. Although results have been obtained upto a depth of 2B, it is clear from the trend that at 3.3B the effect of the strip on SRC would be negligible.

4.6.2 Quantitative Comparison

Laboratory model tests were conducted by *Guido and Sullivan (1985)* on test footings reinforced with square sheets of geotextiles with equal vertical spacing. The goemetry of the model is shown in Fig. 4.13. A square footing of size 0.31m was used. Tests were performed in a square box 1.22m wide with a depth of 0.92m. Square sheets of geotextile were placed concentrically under the footing.

For a single layer of geotextile and for the configuration shown in Fig. 4.13 a reduction in settlement of 17% for a bearing pressure of 60KPa was reported by the researchers. The analytical model gives a settlement reduction of 6.2%. The difference in results is due to the reasons mentioned in art. 5.2. In fact, at lower bearing pressures the settlement reduction from the tests are still lower. Hence at low levels of bearing pressures there will be better agreement between test and analytical results.

A finite element analysis was conducted by *Brown and Poulos (1981)* to study the behaviour of reinforced soils. The definition of their problem is shown in Fig. 4.14. A rigid layer was considered at a depth of 10b (Fig. 4.14). Their analysis gave a settlement reduction of 28% at a stress level of $16q/c'$ for four

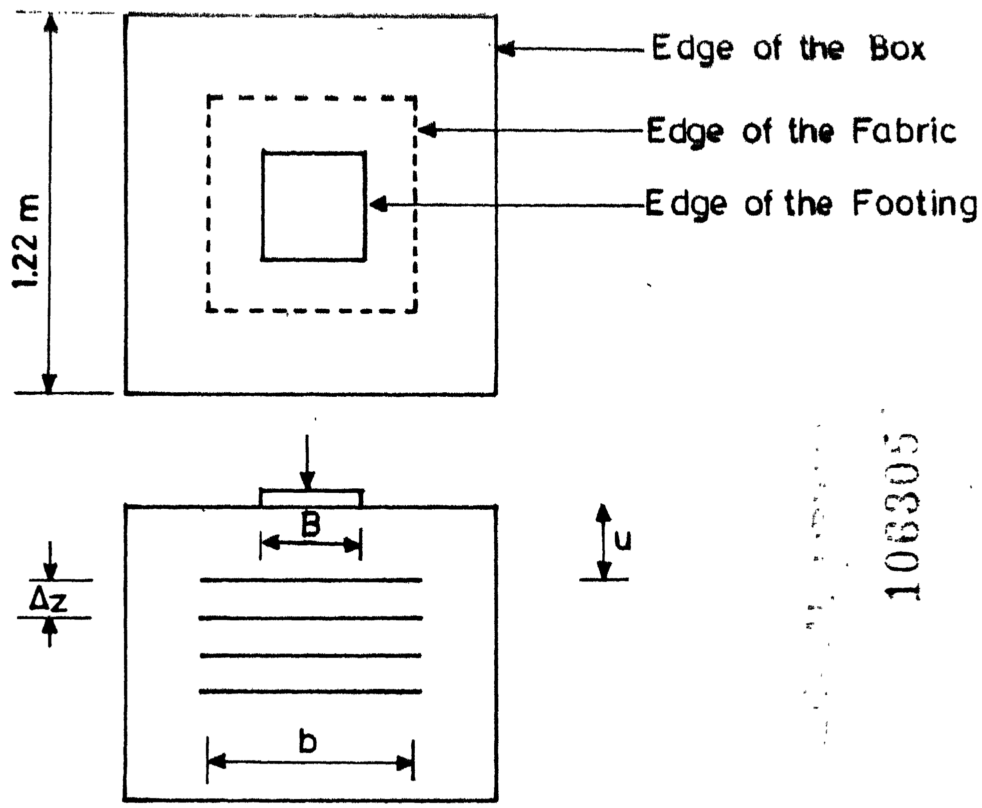

Fig. 4.13 Geometry of the Model (u/B = 0.5, b/B = 2)

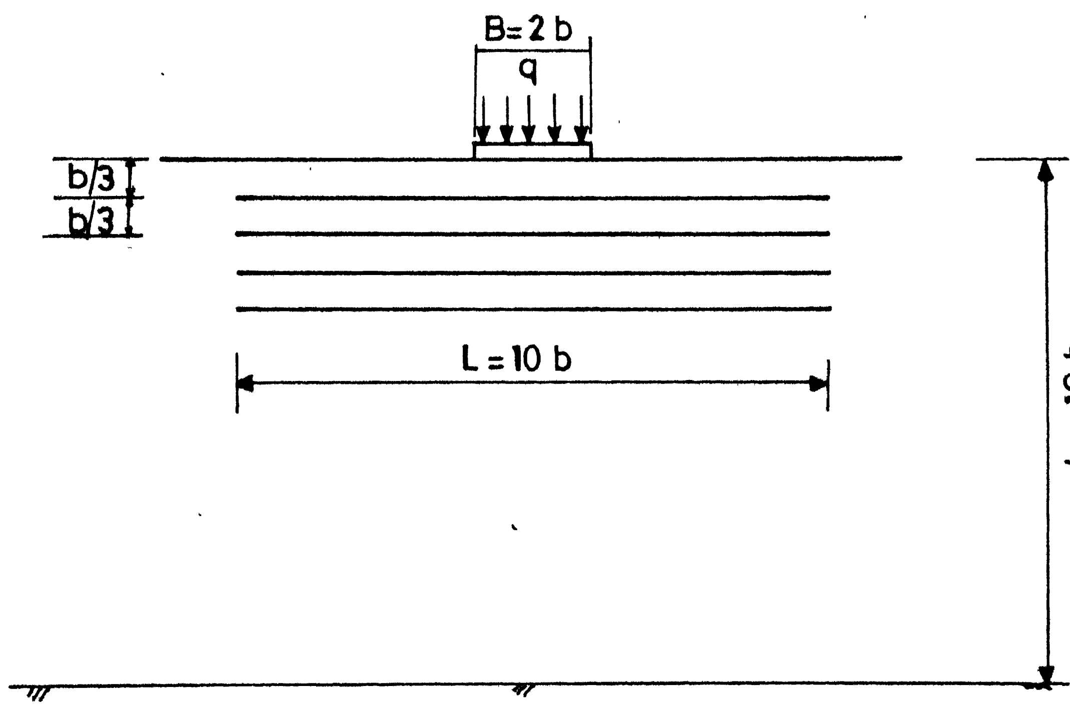

Fig. 4.14 Definition Sketch (Brown and Poulos)

layers of strip reinforcement. The reduction obtained from this analytical model is 2% at the centre of the footing and 5% at its edge. The difference in the results can be explained from the following discussion apart from the reasons given art 5.2.

Brown and Poulos considered an elasto-plastic soil model. They donot say whether the footing is rigid or flexible. The type 5 distribution considered in this thesis is not an exact replica of their stress distribution.

CHAPTER 5
CONCLUSIONS

An elastic continuum approach has been resorted to in modelling the behaviour of strip reinforcements in reducing settlements. Numerical integration of an available elastic solution has been carried out in obtaining the settlement reduction coefficients (SRC) on the surface due to shear stresses acting along the strip. SRC not only at the centre of the footing but also at various points along the footing have been obtained. The parameters affecting the settlement reduction viz., length of the strip, $2l_s$, distance of the strip from the centre of the footing, S_y, and depth of placement of the strip, z_0 have been varied. An optimum depth of placement of the strip based on its length have been specified.

5.1 Conclusions

Based on the results obtained and the discussions in the previous chapter, a broad outline as concluding remarks can be listed as follows:

1. The model is able to bring out qualitatively the effects of strip reinforcements on settlement reduction.

2. An optimal choice on the basic parameters of length, $2l_s$, distance, S_y, and the depth of placement, z_0, of the strip has to be made in order to achieve maximum settlement reduction.

3. For each length of the strip there exists an optimum depth of placement where the effect of the strip on Settlement Réduction Coefficient (SRC) is maximum.

4. Larger the distance of the strip from the point under consideration, lesser is the SRC

5. Increasing the length of the strip beyond a particular length will not improve

the performance of the strip.

6. Smaller the poisson's ratio of the soil, lesser is the SRC. Saturated soils always tend to heave for any stres distribution.

5.2 Limitations

Although this model predicts the behaviour of strip reinforcements qualitatively, it fails to give a correct quantitative picture. The reasons for this aspect can be listed down as follows

1. Only the membrane effect has been considered in this model. It is assumed that the shear atresses are wholly responsible for settlement reduction.
2. The confining effect of the strip on the soil above it is not considered. This confinement can result in further reduction in settlements.
3. The effect of the bearing pressures has not been considered. It is at higher bearing pressures that the reinforcement becomes more effective.
4. The actual stress distribution along the strip is unknown.

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